LONG HILL TOWNSHIP
WASTEWATER TREATMENT PLANT

CAPACITY ASSURANCE REPORT UPDATE

DRAFT

SUBMITTED TO:
TOWNSHIP OF LONG HILL

PREPARED BY:
KLEINFELDER

FEBRUARY 2018

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EXECUTIVE SUMMARY

Background

Long Hill Township (LHT) has been on a voluntary sewer connection ban for many years because wastewater flow to LHT's wastewater treatment plant (WWTP) routinely exceeds its permitted capacity of 0.9 million gallons per day (mgd). While the WWTP successfully treats average flows greater than it was designed to treat, storm events result in significant increases in flow attributable to infiltration and inflow (I&I), to the extent that the hydraulic capacity of the plant is pushed to its limit. Therefore, at the present time the plant cannot handle increases in average flow as the corresponding increase in wet weather flow would cause the plant's hydraulic capacity to be exceeded.

The original Capacity Assurance Report prepared by Kleinfelder in 2010 presented an assessment of the then current wastewater characteristics, the estimated flow rates of infiltration and inflow (I&I) entering the wastewater collection system, and the projected future wastewater characteristics if the voluntary sewer connection ban was lifted. It also evaluated the potential for new or more stringent future NJDEP effluent limits. The findings of this assessment were then used to evaluate the capacity of the existing wastewater treatment plant (WWTP) and the improvements required to accommodate the projected future flows under three (3) system upgrade scenarios involving varying combinations of WWTP improvements and sewer system rehabilitation to reduce I&I: (1) No I&I reduction; (2) 25% I&I reduction; and (3) 50% I&I reduction.

Budgetary capital cost estimates were prepared for the three system upgrade scenarios and are listed below in 2010 dollars.

Budgetary Capital Cost Comparison

System Upgrade Alternative	Budgetary Capital Cost
No I&I Reduction	$4,140,000
25% I&I Reduction	$8,270,000
50% I&I Reduction	$16,760,000

The recommended alternative was the No I&I Reduction alternative. However, because I&I flow rates will increase over time as the system continues to age and deteriorate (referred to as I&I

creep), it was also recommended that sewer rehabilitation improvements be implemented on a regular basis, i.e. analogous to the regular implementation of road re-paving improvements.

Budgetary costs were also developed for improvements to comply with the anticipated future effluent limit of 0.76 mg/L for total phosphorus (TP). The budgetary capital cost estimate in 2010 dollars was $1.2 million and the annual chemical cost for TP removal was estimated to be $82,000 per year based on 2010 chemical costs. Consistent with all other dischargers in the Passaic River Basin, the 0.76 mg/L effluent limit for TP will be imposed upon renewal of the current NJPDES Permit.

LHT decided to defer the implementation of improvements to accommodate future flows and instead to proceed with a project to address two needed improvements at the wastewater treatment plant (replacement of the existing UV disinfection system and influent screening system) and to rehabilitate a relatively small portion of the sanitary sewer (13,162 linear feet out of a total system length of 286,290 linear feet, i.e. 4.6% of the system) in an area designated as high priority based on a prior sewer system flow monitoring program. This project was subsequently designed in 2013, and a construction contract was awarded in 2014. The sewer rehabilitation improvements were completed by January 2015 and the plant improvements were completed by October 2015. The total construction cost was approximately $2.6 million.

The Township is now considering implementing capacity improvement to accommodate future flows, and as the first step in this process, has requested that the Capacity Assurance Report be updated.

The objectives of the Capacity Assurance Report Update are to:

1. Update the current wastewater characteristics including any identifiable change in I&I flow rates following the limited sewer rehabilitation
2. Update the projected future wastewater characteristics
3. Update the assessment of plant capacity based on the updated wastewater characteristics
4. Update the budgetary costs for capacity improvements to accommodate the future flow
5. Update the budgetary costs to comply with the 0.76 mg/L TP limit

Existing Facilities

The WWTP was originally constructed in the 1930s, and has undergone major upgrades in 1975, 1984, 991 and most recently in 2015. The current facilities provide advanced treatment and consist of an influent pumping system, two (2) cylindrical fine screens, two (2) oxidation ditches, two (2) final clarifiers, four (4) effluent filters, a post aeration system, an ultraviolet disinfection system, and a sludge thickening and storage system.

The sanitary sewer system, which delivers wastewater flow to the WWTP, consists of the following components:

- 286,290 Linear Feet (LF) of Township-owned sanitary sewer mains
- 221,325 LF of privately-owned service lateral pipe
- 1,260 manholes
- 8 pumping stations
- 15,200 LF of force mains

Significant portions of the sanitary sewer system are either in or adjacent to flood plains.

Wastewater Characterization

Influent data was obtained for the years 2015, 2016 and 2017 to characterize the key influent parameters relevant to plant capacity. The data was analyzed to determine the average annual, maximum monthly (i.e. highest 30 day average), and maximum daily (i.e. highest 24 hour average) values during each year. The variability in each parameter was characterized by peaking factors, which are calculated as the maximum value divided by the corresponding annual average value. The resulting current wastewater characteristics are summarized in the table below.

Current Wastewater Characteristics

Parameter	Units	Average Annual	Maximum Monthly	MM:AA Peaking Factor	Maximum Daily	MD:AA Peaking Factor	Peak Hourly	MH:AA Peaking Factor
Flow	mgd	1.04	1.91	1.8	3.47	3.3	4.4	4.2
CBOD	mg/l	99	87	-	121	-	-	-
	Lb/d	859	1,381	1.6	3,517	4.1	-	-
TSS	mg/l	117	113	-	198	-	-	-
	Lb/d	1,016	1,805	1.8	5,744	5.7	-	-
TP	mg/l	3.6	3.2	-	5.7	-	-	-
	Lb/d	31	51	2.1	164	5.5	-	-
NH_3 – N	mg/l	11	10	-	14	-	-	-
	Lb/d	98	157	1.6	400	4.1	-	-
TKN – N	mg/l	18	16	-	22	-	-	-
	Lb/d	156	251	1.6	639	4.1	-	-

Current Infiltration and Inflow (I&I)

The hourly flow and daily precipitation data from 2016 were analyzed to calculate the existing flow rates of I&I in the sanitary sewer system, i.e., following rehabilitation of 4.6% of the sewer system which was completed in January 2015. I&I consists of Rainfall Dependent I&I (RDII) and Base Infiltration. RDII occurs as a direct result of rainfall while base infiltration is the result of groundwater entering the system. The current annual average flow rate of I&I was estimated to be approximately 0.59 mgd, comprised of 0.38 mgd of base infiltration and 0.21 mgd of RDII. The current peak flow rate of I&I was estimated to be approximately 3.4 mgd, with RDII accounting for approximately 50%, or 1.7 mgd, of the total peak I&I flow rate.

Based on a comparison of I&I follow rates in 2009 versus 2016, it is concluded that rehabilitating 4.6% of the sanitary sewer system has not resulted in a measurable reduction in sanitary sewer system I&I. It would also appear that I&I creep due to ongoing deterioration of the 95.4% of the system that was not rehabilitated, has more than compensated for the reductions in I&I that may have occurred in the portion of the system that was rehabilitated.

Plant Performance

The existing plant produces effluent concentrations of $CBOD_5$, TSS, NH_3-N, and TP that are significantly below the corresponding current effluent limitations on a monthly average basis. However, these results are based on sampling of most parameters only 3 times per month, and do not reflect the significant difficulties and challenges experienced by the plant during peak wet weather flow events.

While the existing plant complies with the current TP effluent limitations, plant improvements will be required to achieve the future TP limit of 0.76 mg/L

Future Flow

The build-out future average flow was established for the LHT WWTP in the Interim Draft Wastewater Management Plan (WMP) for Morris County. The WMP indicated a build-out average flow of 1.242 mgd which is about 0.2 mgd greater than the current annual average flow of 1.04 mgd.

To estimate the variability in future flows, the current peaking factors were utilized to estimate the future maximum monthly average flow, future maximum daily average flow and future peak hourly flow corresponding to the future annual average flow of 1.24 mgd.

By applying the same peaking factors for future flows as currently exist for current flows, it is inherently assumed that the flow rates of I&I will increase in proportion to the increase in future flows. Therefore, the future flows used in the plant capacity assessment reflect a modest degree of I&I creep in the future. However, substantial I&I creep can be expected to occur unless an ongoing program of sewer rehabilitation is implemented. The table below presents a comparison of the current and future flows.

Summary of Current and Future Flows

Flow Condition	Current	Future
Annual Average	1.04 mgd	1.24 mgd
Maximum Monthly Average	1.8 mgd	2.27 mgd
Maximum Daily Average	3.47 mgd	4.14 mgd
Peak Hourly	4.4 mgd	5.2 mgd

Plant Capacity Evaluation

The capacity of each major component of the WWTP was evaluated to determine its adequacy for the future flows and loads. The plant components with insufficient capacity for future flows are listed below.

1. Influent Pumping System
2. Return Sludge Pumping System
3. Effluent Filters

System Improvement Alternatives for Future Flows

Based on the capacity deficiencies listed above, there are two basic alternatives for WWTP improvements that will enable future development within the sewer service area:

1. Construct a flow equalization system to temporarily store peak wet weather flow in excess of WWTP capacity. This alternative is referred to as the Flow Equalization Alternative.
2. Increase the peak flow capacity of each capacity deficient plant component listed above. This alternative is referred to as the Plant Expansion Alternative

Budgetary capital costs were developed for both alternatives and are summarized in the table below.

Budgetary Capital Cost Comparison

Alternative	Budgetary Capital Cost
Flow Equalization	$4.4 million
Plant Expansion	$2.8 million

In addition to significantly lower cost, the plant expansion alternative also provides permitting benefits because all improvements would occur within existing structures, thereby precluding the need for a Flood Hazard Area Permit as would be required for the flow equalization alternative. Therefore, the recommended alternative to provide capacity for future growth is the plant expansion alternative.

Plant Improvements for Future TP Effluent Limit

To achieve the anticipated future monthly average TP limit of 0.76 mg/L, a coagulant storage and feed system and related improvements must be installed. The budgetary capital cost estimate is $0.8 million.

Based on recent experience at a nearby authority, it is estimated that the average coagulant feed rate will be approximately 80 gallons per day. At the current bulk cost of $5.0 per gallon, the corresponding annual chemical cost would be about $146,000 per year. To more accurately estimate chemical costs, it is re recommended that bench testing be conducted to confirm the optimum coagulant and site-specific dose that will be required to achieve the 0.76 mg/L effluent limit.

The addition of a coagulant will also increase sludge production, typically by about 20, resulting in approximately a 20% increase in current sludge disposal costs.

Conclusions and Recommendations

The key conclusions and recommendations resulting from the Capacity Assurance Report Update are summarized below.

1. There has not been a measurable reduction in sanitary sewer system I&I resulting from the recent sewer rehabilitation project encompassing 4.6% of the overall system.
2. The lowest cost and recommended alternative to provide sufficient capacity for future growth is the plant expansion alternative.

3. The budgetary capital cost estimate for the plant expansion alternative is approximately $2.8 million in 2018 dollars, based on implementation of the following plant improvements:
   a. Replacement of the four (4) existing sand filters with three (3) disc filters to increase peak hour flow capacity of the effluent filters from 2.8 mgd to 5.2 mgd.
   b. Replacement of Influent Pumps #3 and #4 with larger units to increase the firm capacity of the influent pumping system from 3.4 mgd to 5.2 mgd.
   c. Replacement of the four (4) existing return sludge pumps with larger units to increase the firm capacity of the return sludge pumping system from 1.2 mgd to 2.5 mgd.
4. NJDEP will lower the TP effluent limit to 0.76 mg/L when it renews the NJPDES Permit, likely within the next few months. The budgetary capital cost estimate for the improvements needed to achieve compliance with the new TP effluent limit is approximately $0.8 million. Pending site-specific testing to confirm actual coagulant dose, the estimated annual chemical cost to achieve compliance with this limit is approximately $146,000 per year. Coagulant addition will increase sludge generation, typically by approximately 20%, resulting in approximately a 20% increase in the current annual cost of sludge disposal.
5. Without ongoing I&I reduction activities, the low rate of I&I will increase in the future as the wastewater collection system continues to age and deteriorate. Therefore, it is recommended that sewer rehabilitation improvements be implemented on a regular basis, similar to the regular implementation of road re-paving projects.
6. The WWTP's NJPDES Permit will need to be modified to increase the permitted capacity from 0.9 to 1.24 mgd. Because NJPDES Permits must be consistent with the relevant WMP, the LHT WMP will need to be amended before the modified NJPDES Permit will be approved by NJDEP.

1.0 INTRODUCTION

Long Hill Township (LHT) has been on a voluntary sewer connection ban for many years because wastewater flow to LHT's wastewater treatment plant (WWTP) routinely exceeds its permitted capacity of 0.9 million gallons per day (mgd). While the WWTP successfully treats average flows greater than it was designed to treat, storm events result in significant increases in flow attributable to infiltration and inflow (I&I), to the extent that the hydraulic capacity of the plant is pushed to its limit. Therefore, at the present time the plant cannot handle increases in average flow as the corresponding increase in wet weather flow would cause the plant's hydraulic capacity to be exceeded.

The original Capacity Assurance Report prepared by Kleinfelder in 2010 presented an assessment of the then current wastewater characteristics, the estimated flow rates of infiltration and inflow (I&I) entering the wastewater collection system, and the projected future wastewater characteristics if the voluntary sewer connection ban was terminated. It also evaluated the potential for new or more stringent future NJDEP effluent limits. The findings of this assessment were then used to evaluate the capacity of the existing wastewater treatment plant (WWTP) and the improvements required to accommodate the projected future flows under three (3) system upgrade scenarios involving varying combinations of WWTP improvements and sewer system rehabilitation to reduce I&I:

1. No I&I reduction
2. 25% I&I reduction
3. 50% I&I reduction

Budgetary capital cost estimates were prepared for the three system upgrade scenarios and are listed below in 2010 dollars.

Budgetary Capital Cost Comparison

System Upgrade Alternative	Budgetary Capital Cost
No I&I Reduction	$4,140,000
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The recommended alternative to accommodate future flows was the No I&I Reduction alternative. However, because I&I flow rates will increase over time as the system continues to age and deteriorate (referred to as I&I creep), it was also recommended that sewer rehabilitation improvements be implemented on a regular basis, i.e. analogous to the regular implementation of road paving improvements.

Budgetary costs were also developed for improvements to comply with the anticipated future effluent limit of 0.76 mg/L for total phosphorus (TP). The budgetary capital cost estimate in 2010 dollars was $1.2 million and the annual chemical cost for TP removal was estimated to be $82,000 per year based on 2010 chemical costs. Consistent with all other dischargers in the Passaic River Basin, the 0.76 mg/L effluent limit for TP will be imposed upon renewal of the current NJPDES Permit.

LHT decided to defer the implementation of improvements to accommodate future flows and to defer implementation of the phosphorus removal improvement until the new NJPDES Permit is issued. Instead, LHT decided to proceed with a project to address two needed improvements at the wastewater treatment plant (replacement of the existing UV disinfection system and influent screening system) and to rehabilitate a relatively small portion of the sanitary sewer (13,162 linear feet out of a total system length of 286,290 linear feet, i.e. 4.6% of the system) in an area designated as high priority based on a prior sewer system flow monitoring program. This project was subsequently designed in 2013, and a construction contract was awarded in 2014. The sewer rehabilitation improvements were completed by January 2015 and the plant improvements were completed by October 2015. The total construction cost was approximately $2.6 million.

The Township is now considering implementing capacity improvement to accommodate future flows, and as the first step in this process, has requested that the Capacity Assurance Report be updated.

The objectives of the Capacity Assurance Report Update are to:

1. Update the current wastewater characteristics including any identifiable change in I&I flow rates following the limited sewer rehabilitation
2. Update the projected future wastewater characteristics
3. Update the assessment of plant capacity based on the updated wastewater characteristics
4. Update the budgetary costs for capacity improvements to accommodate the future flow
5. Update the budgetary costs to comply with the 0.76 mg/L TP limit

2.0 EXISTING FACILITIES

The LHT WWTP was originally constructed in the 1930s, and has undergone updates in 1975, 1984, 1991, and most recently in 2015. The current facilities provide advanced treatment and consist of an influent pumping system, two (2) cylindrical fine screens, two (2) oxidation ditches, two (2) final clarifiers, four (4) effluent filters of the upflow continuous backwash type, a post aeration system, an ultraviolet disinfection system, and a sludge thickening and storage system. The principal treatment facility components are summarized in Table 1.

Table 1: Existing Facilities

Unit Process	Component	# of Units	Description
Headworks	Influent Pump Station	1	25-foot deep well with submersible pumps that lift influent 40 feet to static screens.
	Influent Submersible Pumps	4	Varying capacity at 15 hp, 20 hp, and 44 hp.
	Cylindrical Fine Screens	2	Tank mounted fine screens with 3/8" screen openings and a peak flow capacity of 5 mgd per screen
Oxidation Ditches	Distribution Chamber #1	1	Concrete box with wood baffle, (2) aluminum slide gates, and (2) 16" outlet pipes.
	Oxidation Ditch #1	1	Tank volume is approximately 293,000 gallons with dimensions 174'L x 14'W x 12' SWD. (2) 14'L Lakeside brush aerators supply oxygen at a rate of 6.6 lbs O_2/hr/ft of rotator length.
	Oxidation Ditch #2	1	Tank volume is approximately 614,000 gallons with dimensions 165'L x 45'W x 12' SWD. (2) 21' L Envirodyne brush aerators provide oxygen at a rate of 5.85 lbs O_2/hr/ft of rotator length.
Secondary Clarifiers	Distribution Chamber #2	1	Concrete distribution box with (2) 16" outlet pipes.
	Clarifiers	2	50' diameter half bridge clarifiers, with a SWD of 11'8" and a surface area of 3,927 sq. ft each.
	Return Activated Sludge Pumps	4	10 hp RAS pumps with variable frequency drives; each rated for 425 gpm (0.61 mgd) at 25' of head.
	Waste Activated Sludge Pumps	2	WAS pumps are each rated for 470 gpm (0.68 mgd) at 12'of head.
Filters	Continuous Backwash Sand Filters	4	Parkson Dynasand continuous backwash filters; each unit has a filtration area of 150 ft^2.
Post-Aeration	Aeration Tank	1	Concrete tank with two (2) 220 cfm air blowers and 28 coarse bubble diffusers spaced at 2' intervals.
Disinfection	Ultraviolet Disinfection System	1	Trojan Technologies low pressure high intensity UV disinfection system in two channels, each with a combined maximum daily flow capacity of 4.4 mgd
Sludge Handling	Sludge Storage Facilities	2	(2) 25' diameter aerated, concrete tanks with 27' SWD. Total useable storage volume approximately 150,000 gallons. Mechanical thickener located in Digester Building between tanks.

A Site Plan and Flow Schematic are presented in Figures 1 and 2, respectively and show the physical arrangement of treatment facilities and how wastewater flows through the plant.

LHT's sanitary sewer collection system, which delivers wastewater flow to the WWTP, consists of the following components:

- 286,290 Linear Feet (LF) of Township-owned sanitary sewers:
- 221,325 LF of privately-owned service lateral pipe
- 1,260 manholes
- 8 pumping stations
- 15,200 LF of force mains

A portion of the system dates to the 1930's and 1940's, which coincided with the date of the original wastewater treatment plant. Significant additions to the collection system occurred in the 1970's, coinciding with the construction-grants era and upgrades to the original WWTP. A map of the sanitary sewer collection system with 100-year flood plains is presented in Figure 3. As indicated, significant portions of the sanitary sewer system are either in or adjacent to flood plains.

3.0 CURRENT AND FUTURE EFFLUENT LIMITATIONS

The LHT WWTP NJPDES Permit (NJ0024465) has an effective date of February 1, 2006, and an expiration date of January 31, 2011. However, because NJDEP has not yet renewed the NJPDES Permit, the existing NJPDES Permit remains in effect. Table 2 summarizes the current key effluent limitations related to plant capacity.

Table 2: Long Hill Township WWTP Current Key Effluent Limitations

Parameter	**Average Month**		**Maximum Weekly**	
Carbonaceous BOD	8 mg/L	27 kg/day	12 mg/L	41 kg/day
TSS	30 mg/L	100 kg/day	45 mg/L	150 kg/day
NH_3-N (May through Oct.)	2 mg/L	6.8 kg/day	3mg/L	10.2 kg/day
NH_3-N (Nov. through April)	34.2 mg/L	116 kg/day	N/A	N/A
Total Phosphorus (May through Oct.)	4.4 mg/L	N/A	N/A	N/A
Total Phosphorus (Nov. through April)	3.7 mg/L	N/A	N/A	N/A
Fecal Coliform	200 col/100 ml	N/A	400 col/100 ml	
Chlorine Produced Oxidants	0.1 mg/L daily maximum		0.1 kg/day daily maximum	
Dissolved Oxygen	6.0 mg/L weekly minimum			
pH	6.0 minimum, 9.0 maximum			

FIGURE 1:
SITE PLAN

Long Hill WWTP
Long Hill Township
Morris County, NJ

KLEINFELDER
Bright People. Right Solutions.
321 Wall Street, Princeton, NJ 08540
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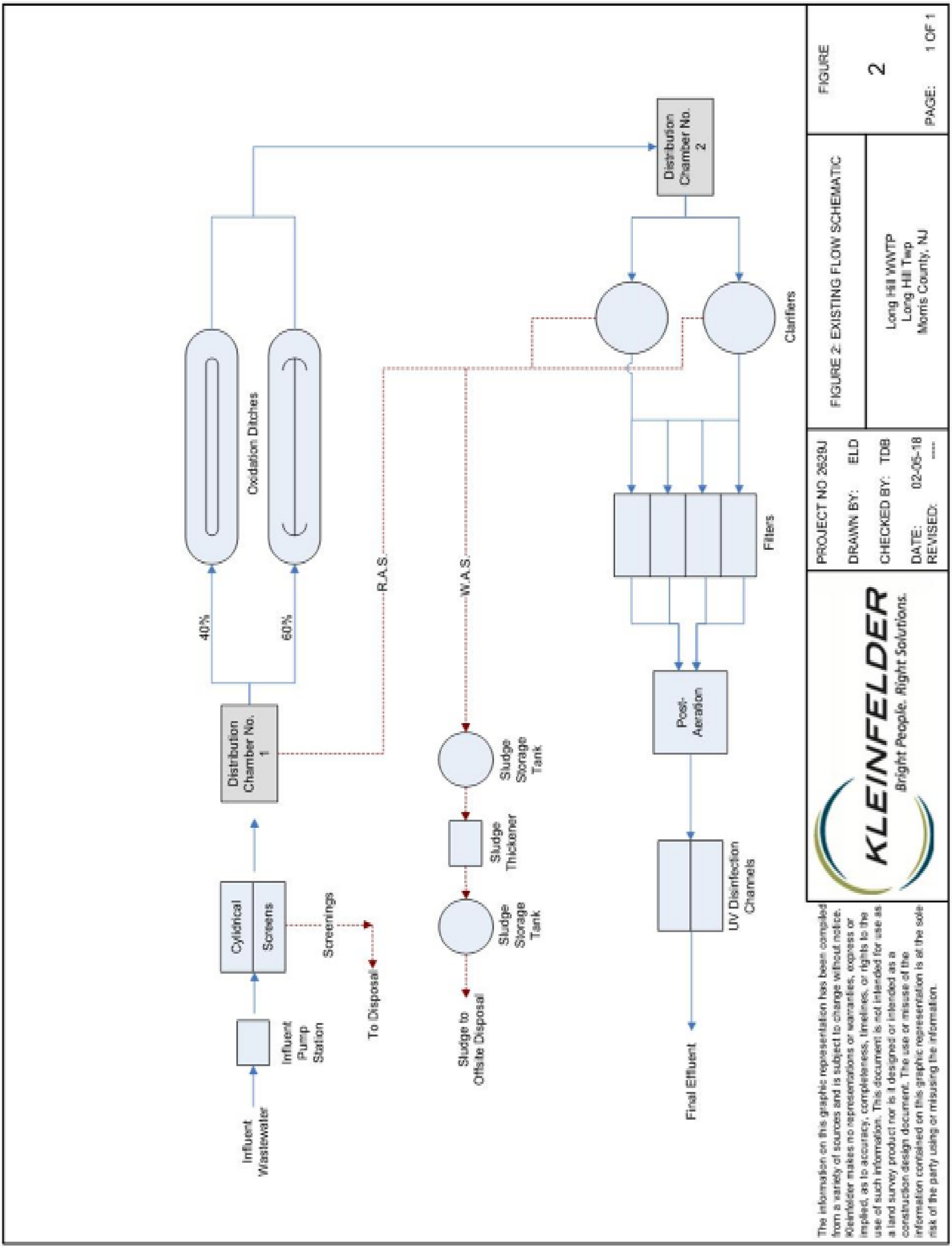
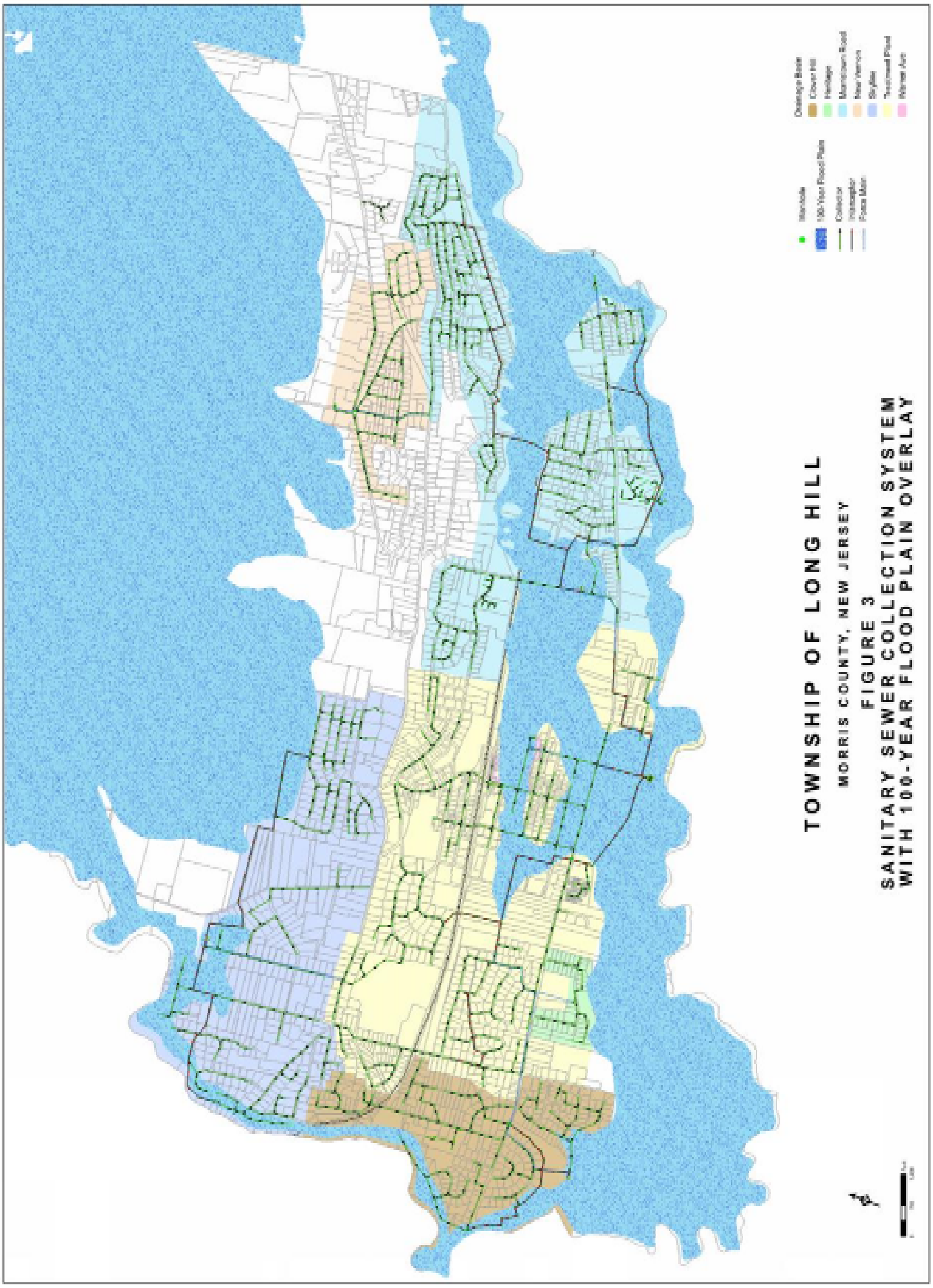
Fence - Approx. WWTP Boundary

Sources:
2018 ESRI Basemaps

PROJECT #:	2629J
DRAWN:	February 2018
DRAWN BY:	ELD
CHECKED BY:	TB
FILE NAME:	

0 100 200
Feet


Influent Wastewater
Influent Pump Station
Cylindrical Screens
Screenings
To Disposal
Distribution Chamber No. 1
40%
60%
Oxidation Ditches
Distribution Chamber No. 2
R.A.S.
W.A.S.
Clarifiers
Filters
Post-Aeration
UV Disinfection Channels
Final Effluent
Sludge Storage Tank
Sludge Thickener
Sludge Storage Tank
Sludge to Offsite Disposal
KLEINFELDER
Bright People. Right Solutions.
PROJECT NO 2628J
DRAWN BY: ELD
CHECKED BY: TDB
DATE: 02-05-18
REVISED: ----
FIGURE 2: EXISTING FLOW SCHEMATIC
Long Hill WWTP
Long Hill Twp
Morris County, NJ
FIGURE
2
PAGE: 1 OF 1


TOWNSHIP OF LONG HILL
MORRIS COUNTY, NEW JERSEY
FIGURE 3
SANITARY SEWER COLLECTION SYSTEM
WITH 100-YEAR FLOOD PLAIN OVERLAY

With regard to future effluent limitations, as described in the original Capacity Assurance Report, NJDEP will impose monthly average TP effluent limits of 0.76 mg/L for all dischargers to the Passaic River. This process has already begun and is occurring in conjunction with renewals of NJPDES Permits. LHT will receive this effluent limit when its NJPDES Permit is renewed, which should be soon considering that LHT current NJPDES Permit is pending renewal at this time. The renewed NJPDES permit will likely establish a three (3) year schedule to achieve compliance.

4.0 WASTEWATER CHARACTERIZATION

Influent data was obtained from Discharge Monitoring Reports (DMRs) for the years 2015, 2016 and 2017 to characterize the key influent parameters relevant to plant capacity, which are flow, Carbonaceous Biochemical Oxygen Demand (CBOD), Total Suspended Solids (TSS) and Total Phosphorus (TP). Influent ammonia nitrogen (NH_3) and influent Total Kjeldahl Nitrogen (TKN) data are not available, as the NJPDES Permit does not require that the influent wastewater be analyzed for these parameters. The data was analyzed to determine the average annual, maximum monthly (i.e. highest 30 day average), and maximum daily (i.e. highest 24 hour average) values during each year. The variability in each parameter was characterized by peaking factors, which are calculated as the maximum value divided by the corresponding annual average value.

Table 3 on the following page summarizes the resulting wastewater characterization data during the years 2015, 2016 and 2017, as well as the average for the three (3) year period.

Consistent with the original Capacity Assurance Report, to establish the current wastewater characteristics for the Capacity Assurance Report Update, the overall average values presented in Table 3 were utilized.

As previously indicated, the plant's NJPDES Permit does not require the reporting of the influent concentration of either NH_3 or TKN. Therefore, current concentrations of these parameters as presented in Table 3 were estimated based on their typical correlation with the influent CBOD concentration (8.8 CBOD:1 $NH\text{-}_3$, 5.5 CBOD:1 TKN).

Table 3: DMR Wastewater Characterization Data Summary 2015, 2016 and 2017

Year	DMR Parameter Description abbrv.	Calculation Type	Average Annual	Maximum Monthly	MM:AA Peaking Factor	Maximum Daily	MD:AA Peaking Factor
2015	Flow, In Conduit or Thru Treatment Plant	Flow (mgd)	1.00	1.90	1.9	3.50	3.5
	BOD, Carbonaceous 5 Day, 20oC	Concentration (mg/L)	95	146	1.5	228	2.4
		Load (lb/d)	732	1,026	1.4	3,338	4.6
	Solids, Total Suspended	Concentration (mg/L)	131	276	2.1	380	2.9
		Load (lb/d)	990	1,305	1.3	4,965	5.0
	Phosphorus, Total (as P)	Concentration (mg/L)	4.2	11.5	2.8	13.5	3.2
		Load (lb/d)	33.8	115.2	3.4	259.1	7.7
2016	Flow, In Conduit or Thru Treatment Plant	Flow (mgd)	1.00	2.10	2.1	3.40	3.4
	BOD, Carbonaceous 5 Day, 20oC	Concentration (mg/L)	116	185	1.6	232	2.0
		Load (lb/d)	896	1,346	1.5	2,882	3.2
	Solids, Total Suspended	Concentration (mg/L)	117	150	1.3	273	2.3
		Load (lb/d)	953	1,472	1.5	3,417	3.6
	Phosphorus, Total (as P)	Concentration (mg/L)	3.7	5.3	1.4	5.9	1.6
		Load (lb/d)	28.4	39.1	1.4	99.3	3.5
2017	Flow, In Conduit or Thru Treatment Plant	Flow (mgd)	1.12	1.69	1.5	3.50	3.1
	BOD, Carbonaceous 5 Day, 20oC	Concentration (mg/L)	106	194	1.8	233	2.2
		Load (lb/d)	950	1,821	1.9	4,277	4.5
	Solids, Total Suspended	Concentration (mg/L)	125	307	2.5	410	3.3
		Load (lb/d)	1,106	2,725	2.5	9,237	8.4
	Phosphorus, Total (as P)	Concentration (mg/L)	3.6	6.7	1.9	8.9	2.5
		Load (lb/d)	31.3	51.4	1.6	163.9	5.2
Overall Average	Flow, In Conduit or Thru Treatment Plant	Flow (mgd)	1.04	1.90	1.83	3.47	3.33
	BOD, Carbonaceous 5 Day, 20oC	Concentration (mg/L)	105	175	1.66	23	2.21
		Load (lb/d)	859	1,398	1.61	3,499	4.09
	Solids, Total Suspended	Concentration (mg/L)	124	244	1.95	354	2.84
		Load (lb/d)	1,016	1,834	1.78	5,873	5.65
	Phosphorus, Total (as P)	Concentration (mg/L)	3.8	7.8	2.02	9.5	2.44
		Load (lb/d)	31	68	2.14	174	5.46

With regard to peak hourly flow, because the plant no longer records peak hourly flow, it was assumed that the ratio of peak hourly flow to maximum daily flow that existed at the time the original Capacity Assurance Report was developed (1.28) is the same ratio that currently exists.

Therefore, the current peak hourly flow was estimated by multiplying the current maximum daily flow of 3.47 mgd by 1.28, resulting in a current peak hourly flow of 4.4 mgd.

The resulting Current Wastewater Characteristics are presented in Table 4.

Table 4: Current Wastewater Characteristics

Parameter	Units	Average Annual	Maximum Monthly	MM:AA Peaking Factor	Maximum Daily	MD:AA Peaking Factor	Peak Hourly	MH:AA Peaking Factor
Flow	mgd	1.04	1.91	1.8	3.47	3.3	4.4	4.2
CBOD	mg/l	99	87	-	121	-	-	-
	lb/d	859	1,381	1.6	3,517	4.1	-	-
TSS	mg/l	117	113	-	198	-	-	-
	lb/d	1,016	1,805	1.8	5,744	5.7	-	-
TP	mg/l	3.6	3.2	-	5.7	-	-	-
	lb/d	31	51	2.1	164	5.5	-	-
$NH_3 - N$	mg/l	11	10	-	14	-	-	-
	lb/d	98	157	1.6	400	4.1	-	-
TKN – N	mg/l	18	16	-	22	-	-	-
	lb/d	156	251	1.6	639	4.1	-	-

The concentrations presented in Table 4 are equivalent to the load divided by the flow and do not represent the concentration reported in the DMRs.

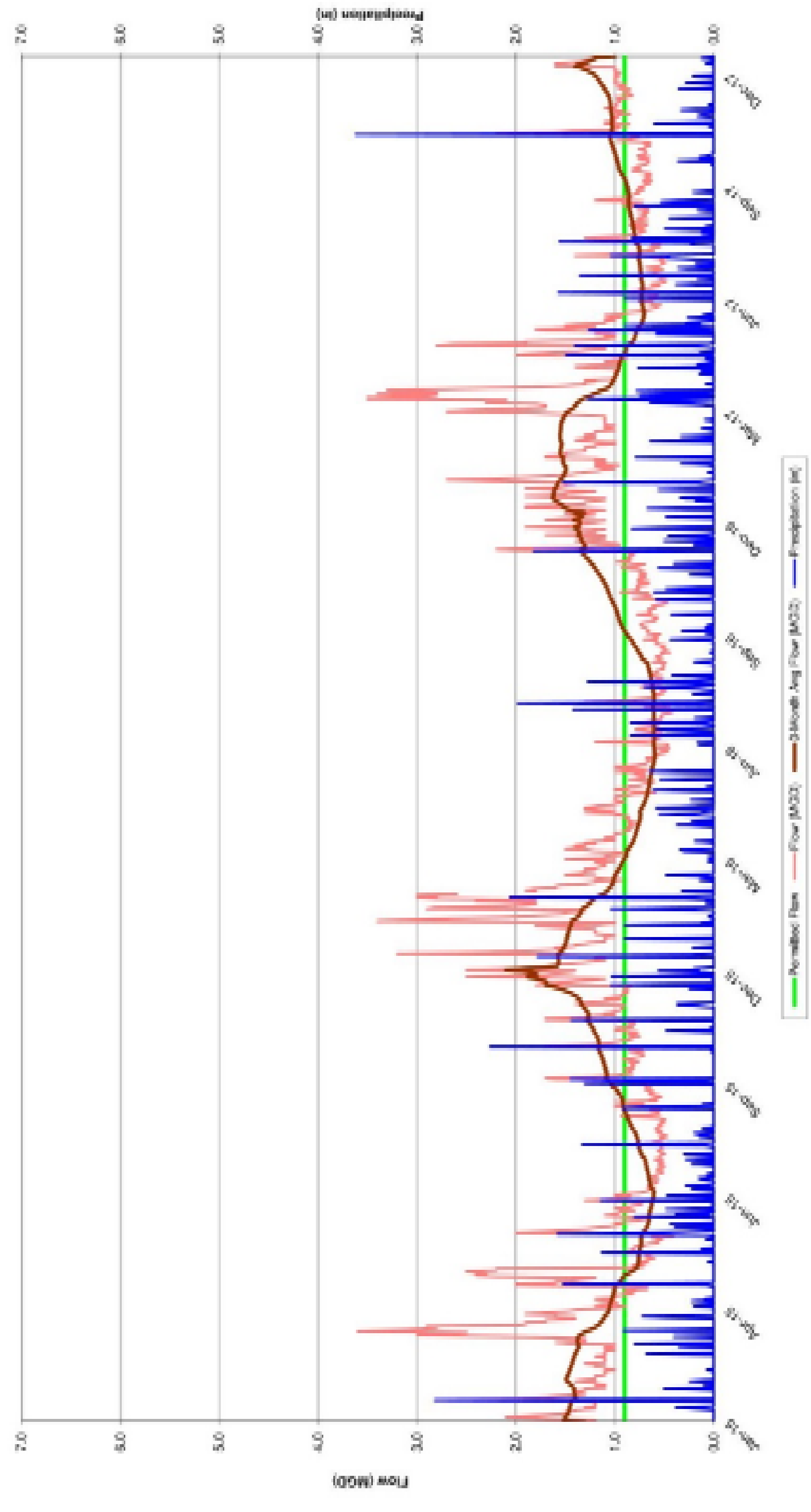
Concentration and loads are not presented for the peak hourly flow, because the peak hourly flow is only used for hydraulic capacity assessment.

5.0 CURRENT INFILTRATION AND INFLOW

The daily flow and precipitation data from 2016 were analyzed to estimate the existing flow rates of I&I in the LHT sewer system.

The year 2016 was selected for this evaluation because it had the highest flow peaking factors during the 3-year period following the sewer rehabilitation project. The daily flow and daily precipitation are shown in Figure 4.

Figure 4. Long Hill WWTP Flow and Precipitation Chart: 2015-2017



Consistent with the methodology utilized in the original Capacity Assurance Report, to evaluate the current I&I, the 2016 flow data was first divided into dry-day flows and wet-day flows. A dry-day was defined as a day in which there was no rainfall and which the five (5) prior days had rainfall amounts less than shown below:

Prior Days	Rainfall (in)
1 day	0.1
3 days	0.4
5 days	1.0

A wet day was any day that did not meet the criteria for a dry-day. The resulting dry day average flows and wet-day average flows during each month in 2016 are presented in Table 5.

Table 5: 2016 Dry-Day and Wet-Day Monthly Average Flows

Month	Average Flow (mgd)	
	Dry-Day	Wet-Day
January	1.28	1.59
February	1.70	2.27
March	1.25	1.32
April	1.00	1.10
May	0.78	1.03
June	0.62	0.81
July	0.50	0.70
August	0.52	0.61
September	0.56	0.62
October	0.61	0.73
November	0.78	0.97
December	1.29	1.32
Annual Average	0.91	1.59

The difference between dry-day average flow and wet-day average flow is the I&I associated with rainfall. The term for this component of I&I is “Rainfall Dependent Infiltration and Inflow” or “RDII”.

The total I&I in the system is comprised of two components: RDII and Base Infiltration. Base infiltration is the result of groundwater, rather than rainwater, entering the system. Base infiltration varies from month to month due to seasonal changes in groundwater levels. The lowest dry-day flows are observed during the summer months of July, August, and September when groundwater levels and thus base infiltration are the lowest.

Based on the information presented in Table 5, the summer average dry-day flow in 2016 was equal to 0.53 mgd (i.e. the average of the monthly average flows in July, August and September)

Because dry days are not influenced by RDII and summer months have the lowest groundwater levels, the summer average dry-day flow is representative of wastewater flow to the WWTP not impacted by I&I. Based on a 2010 Census population of 8,702 and a wastewater flow of 0.53 mgd, the resulting wastewater flow per capita is 60 gal/day, which is well within the expected literature range for domestic wastewater flows exclusive of I&I. Therefore, LHT's current average wastewater flow, exclusive of I&I, is estimated to be 0.53 mgd.

As also shown in Table 5, the average dry-day flow in 2016 was 0.91 mgd. The difference between the annual average dry-day flow (0.91 mgd) and the wastewater flow without I&I (0.53 mgd), represents the annual average base infiltration rate, which in 2016 was 0.38 mgd (0.91 – 0.53 = 0.38). Based on this same methodology in the original Capacity Assurance Report, the annual average base infiltration rate in 2009 was estimated to be 0.25 mgd, which is 0.13 mgd less than in 2016.

In addition to the annual average base infiltration rate of 0.38 mgd, significant RDII also enters the system. As previously indicated, RDII is the extraneous flow that enters a sewer system during and after a rain storm or snow melt. On a monthly average basis, RDII is equal to the monthly average wet day flow minus the corresponding monthly average dry-day flow.

Table 6 summarizes the average RDII and average Base Infiltration during each month of 2016, as well as the annual average.

Table 6: 2016 Monthly Average Wastewater Flow, Base Infiltration and RDII (mgd)

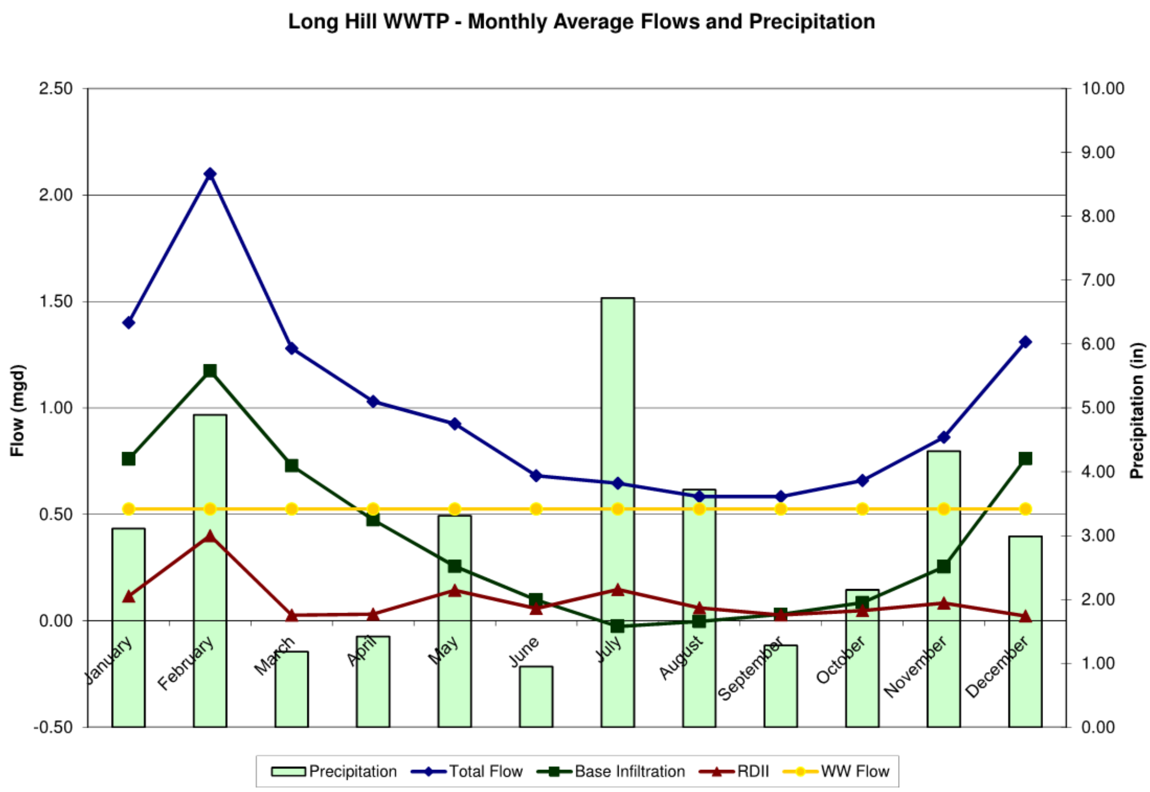
Month	WW Flow	Base Infiltration	RDII	Total
January	0.53	0.76	0.12	1.40
February	0.53	1.17	0.40	2.10
March	0.53	0.73	0.03	1.28
April	0.53	0.47	0.03	1.03
May	0.53	0.26	0.14	0.93
June	0.53	0.10	0.06	0.68
July	0.53	-0.03	0.15	0.65
August	0.53	0.00	0.06	0.58
September	0.53	0.03	0.03	0.58
October	0.53	0.09	0.05	0.66
November	0.53	0.25	0.08	0.86
December	0.53	0.76	0.02	1.31
Average	0.53	0.38	0.09	1.00

Table 7:2016 I&I Components of Annual Average Flow

Flow Type	Flow Rate (mgd)
Summer Average Dry-Day Flow	0.53
Annual Average Base Infiltration Rate	0.38
Annual Average RDII	0.09
Total Annual Average Flow	1.00

Figure 7 below shows the variation during each month of 2016 in total precipitation, monthly average total plant flow, monthly average base infiltration and monthly average RDII.

Figure 5: Breakdown of Monthly Average Flows and Precipitation



As shown in Table 6, Table 7 and Figure 5, on an average annual and monthly average basis, base infiltration contributes more extraneous flow to the sewer system than does RDII. However, this is not the case during peak flow events, as further discussed below.

The highest maximum daily flow during 2016 was 3.4 mgd which occurred on February 6th. Based on the method described in Section 4.0, the corresponding peak hourly flow is estimated to be 4.4 mgd. This event was a result of rainfall which totaled 1.02 inches over a 3-day period during which there was also snow melt, however the extent of snow melt is not known. The base infiltration and RDII components of these maximum day and peak hourly flows are estimated as follows.

The base infiltration rate during the maximum day and peak hourly flows was estimated by subtracting the summer average dry-day flow (0.53 mgd) from the February 2016 average dry-day flow (1.70 mgd), resulting in a base infiltration rate of 1.17 mgd. The RDII component of maximum day flow was estimated by subtracting the summer average dry-day flow (0.53 mgd) and February base infiltration (1.17 mgd) from the February 6th total maximum day flow (3.4 mgd), resulting in a maximum day RDII flow of 1.7 mgd. Similarly, the RDII component of peak hourly flow was estimated by subtracting the summer average dry-day flow (0.53 mgd) and February base infiltration (1.17 mgd) from the February 6th peak hourly flow (4.4 mgd), resulting in a maximum peak hourly RDII flow of 2.7 mgd. This information is summarized in Table 8

Table 8: I&I Components of Peak Flow

Flow Type	Maximum Day (mgd)	Peak Hour (mgd)
Summer Average Dry-Day Flow	0.53	0.53
February 2016 Base Infiltration Rate	1.17	1.17
Peak Event RDII	1.70	2.7
Total Peak Event Flow	3.40	4.4

Therefore, based on the information presented in Table 8, the RDII component of maximum day and peak hourly flows is more significant than base infiltration, accounting for approximately 60% of the maximum daily I&I flow and approximately 70% of the peak hourly I&I flow.

The information presented in Table 8 reflects the sanitary sewer system's current response (i.e. following the sewer rehabilitation project completed in January 2015) to a 1.02-inch rainfall event over three (3) days combined with some snowmelt.

As described in the original Capacity Assurance Report, the sanitary sewer system's response to a 3.2-inch rainfall event over five (5) days in May 2009 without snowmelt resulted in a maximum day flow of 3.2 mgd, a peak hourly flow of 3.9 mgd, a maximum daily I&I flow rate of 2.35 mgd, a peak hourly I&I flow rate of 2.73 mgd and a monthly average base infiltration rate of 0.21 mgd.

Therefore, a 1 inch rain event combined with snow melt in 2016 following sewer rehabilitation resulted in a higher maximum daily flow, peak hourly flow, and monthly average base infiltration rate than a 3.9 inch rain event without snowmelt in 2009, while the peak hourly I&I flow rates were essentially the same.

Accordingly, it is concluded that rehabilitating 4.6% of the sanitary sewer system has not resulted in a measurable reduction in sanitary sewer system I&I. It would also appear that I&I creep due to ongoing deterioration of the 95.4% of the system that was not rehabilitated, has more than compensated for any reductions in I&I that may have occurred in the portion of the system that was rehabilitated.

6.0 PLANT PERFORMANCE

Existing plant performance was characterized by the effluent concentration of the key parameters related to capacity, i.e. $CBOD_5$, TSS, and NH_3-N. From 2015 to 2017, the average annual effluent $CBOD_5$, TSS and NH_3-N, concentrations were 2.2 mg/L, 1.1 mg/L, and 0.1 mg/L, respectively, which are all substantially lower than the NJPDES Permit effluent limitations.

The data was also analyzed for the maximum monthly average, and maximum daily average concentrations, as presented in Table 9. Based on a comparison of the key effluent limitations presented in Table 2 versus the effluent concentrations presented in Table 9, the existing plant produces effluent concentrations of $CBOD_5$, TSS, NH_3-N that are significantly below the corresponding effluent limitations during maximum month and maximum day conditions.

As also shown in Table 9 on the following page, the plant produces annual average effluent TP concentrations ranging from 2.3 mg/L to 2.6 mg/L. Therefore, plant upgrades will be required to achieve the future TP limit of 0.76 mg/L

7.0 FUTURE FLOW

The build-out future average flow was established for the LHT WWTP in the Interim Draft Wastewater Management Plan (WMP) for Morris County. The WMP indicated a build-out average flow of 1.242 mgd which is about 0.2 mgd greater than the current annual average flow of 1.04 mgd from Table 4.

Table 9: 2015-2017 Effluent Concentrations

Year	DMR Parameter Description abbrv.	Average Annual	Maximum Monthly	Maximum Daily
2015	BOD, Carbonaceous 5 Day, 20oC	2.3	3.3	6.0
	Solids, Total Suspended	1.1	3.5	7.0
	Nitrogen, Ammonia Total (as N)**	0.3	1.2	2.3
	Phosphorus, Total (as P)	2.5	3.8	4.1
2016	BOD, Carbonaceous 5 Day, 20oC	2.1	3.0	5.0
	Solids, Total Suspended	1.1	1.5	3.0
	Nitrogen, Ammonia Total (as N)	0.1	0.3	0.5
	Phosphorus, Total (as P)	2.6	4.0	4.3
2017	BOD, Carbonaceous 5 Day, 20oC	2.1	2.7	4.0
	Solids, Total Suspended	1.0	1.8	3.0
	Nitrogen, Ammonia Total (as N)	0.1	0.3	0.5
	Phosphorus, Total (as P)	2.3	3.5	3.6
Average	BOD, Carbonaceous 5 Day, 20oC	2.2	3.0	5.0
	Solids, Total Suspended	1.1	2.3	4.3
	Nitrogen, Ammonia Total (as N)	0.1	0.6	1.1
	Phosphorus, Total (as P)	2.5	3.8	4.0

To estimate the variability in future flows and loads, the current peaking factors from Table 4 will be utilized, while also assuming that the annual average wastewater strength remains general consistent with the current strength.

The resulting future flows and loads are presented in Table 10 on the following page.

By applying the same peaking factors for future flows as currently exist for current flows, it is inherently assumed that the flow rates of I&I will increase in proportion to the increase in future flows. Therefore, the future flows presented in Table 10 reflect a modest degree of I&I creep in the future. However, substantial I&I creep can be expected to occur unless an ongoing program of sewer rehabilitation is implemented, akin to ongoing road re-paving projects to maintain an appropriate degree of road condition throughout the Township.

Table 10: Future Flows and Loads

Parameter	Units	Average Annual	Maximum Monthly	MM:AA Peaking Factor	Maximum Daily	MD:AA Peaking Factor	Maximum Hourly	MH:AA Peaking Factor
Flow	mgd	1.242	2.27	1.8	4.14	3.3	5.24	4.2
CBOD	mg/l	99	103	-	121	-	-	-
	lb/day	1,024	1,645	1.6	4,190	4.1	-	-
TSS	mg/l	117	113	-	198	-	-	-
	lb/day	1,210	2,149	1.8	6,841	5.7	-	-
TP	mg/l	3.6	3.2	-	5.9	-	-	-
	lb/day	37	80	2.1	204	5.5	-	-
NH3-N	mg/l	11	10	-	13.8	-	-	-
	lb/day	116	187	1.6	476	4.1	-	-
TKN-N	mg/l	18	16	-	22.0	-	-	-
	lb/day	186	299	1.6	762	4.1	-	-

The original Capacity Assurance Report also presented future flows for hypothetical 25% and 50% reductions in I&I. However, the original Capacity Assurance Report also concluded that reduction in I&I through sewer rehabilitation is not a cost effective approach to providing capacity to accommodate future flows. Therefore, this Capacity Assurance Report Update does not present future flows for hypothetical reductions in I&I.

8.0 PLANT CAPACITY EVALUATION

This section of the report evaluates the adequacy of each major component of the plant under the future flow and load scenario presented in Table 10. To evaluate each treatment component of the plant, detailed flow and mass balances were developed that:

- Present the average and maximum influent flows and loads through the plant.
- Provide physical information regarding each component (such as tank dimensions).
- Identify key sizing/capacity related criteria for each unit process (such as detention time, surface overflow rate, etc.).
- Evaluate conformance with the relevant sizing criteria at average and maximum conditions.
- Generate essential data, such as oxygen requirements and sludge production rates, required to evaluate capacity adequacy.
- Project expected effluent quality for CBOD, TSS and NH_3, based on calibration of existing performance to key process control parameters.

- Enables an evaluation of how changes in key control parameters, such as RAS flow rate, solids retention time, and dissolved oxygen concentration in the oxidation ditches impacts the process.

For evaluation of the oxidation ditches, a kinetic analysis was also performed. The plant components related solely to hydraulic capacity, such as the influent pumps and influent screens, are not presented in the flow and mass balances but rather are discussed separately below. The following key plant components were evaluated:

- Influent Pumping System
- Screening
- Oxidation Ditches
- Final Clarifiers and Return Sludge Pumping System
- Waste Sludge Pumping System
- Effluent Filters
- Post Aeration System
- UV Disinfection System
- Sludge Storage System

8.1 Influent Pumping System

Influent pumping systems are sized to provide “firm capacity” for peak hourly flows. The “firm capacity” is the pumping capacity that exists when one pump is out of service. If the pumping system includes multiple size pumps, the “firm capacity” is the capacity that exists when one of the largest pumps is out of service.

The existing influent pumping station consists of a 25-foot deep well with four (4) submersible pumps that lift the wastewater approximately 40 feet to the influent screens. The following submersible pumps are located at the influent pumping station:

- Pump #1: Flygt Model CP3140 with 15 hp motor
- Pump #2: Flygt Model CP3152 with 20 hp motor
- Pumps #3 & #4: KSB Model KRTK 200-400/226 with 44 hp motors and trimmed impellers

During very high flow events, pump #1, #3, and #4 are all in operation, and the observed capacity is nominally greater than 4 mgd. However, since this capacity requires that both of the largest pumps be in operation, it cannot be considered the reliable firm capacity. With one of the largest pumps out of service, the capacity is about 3.4 mgd. Therefore, the firm capacity of the existing influent pumping system is approximately 3.4 mgd, which is less than the current peak hourly flow of 4.4 mgd and future peak hourly flow of 5.2 mgd.

To provide a firm capacity of 5.2 mgd for the future peak hourly flow, Pump #3 and #4 will need to be replaced with larger units.

8.2 Screening

Influent screens remove debris from the wastewater that could otherwise clog or damage downstream equipment or processes. Influent screens are sized for peak hourly flows.

The new influent cylindrical fine screens were manufactured by Huber and were installed in 2015 to replace the original static screens. Each of the two (2) cylindrical fine screens has a peak flow capacity of 5.0 mgd. Therefore, the existing screening system has sufficient capacity for future flows.

8.3 Oxidation Ditches

Oxidation ditches are biological reactors that provide an environment suitable for the growth of microorganisms which remove CBOD and NH_3 from the wastewater. Oxidation ditches, and their associated mechanical aeration equipment, are sized based primarily on two criteria:

1. Aeration equipment capacity sufficient to supply the required oxygen for CBOD and NH_3 removal.
2. Tank volume sufficient to hold the mass of microorganisms needed to remove the CBOD and NH_3 while also providing the appropriate environmental conditions for microorganisms to perform properly.

These criteria are addressed separately below.

8.3.1 Aeration Equipment

When assessing the capacity of aeration equipment, the wastewater oxygen requirement is first calculated based on the pounds of oxygen required per pound of BOD and NH_3 removed. The wastewater oxygen requirement is then converted to a standard oxygen requirement (SOR) based on site specific conditions of temperature, operating dissolved oxygen (DO) concentration, alpha coefficient (i.e. the ratio of wastewater oxygen transfer to clean water oxygen transfer) and beta coefficient (i.e. the salinity correction factor). Aeration devices are then evaluated with respect to their ability to supply the required SOR.

As shown in Table 1, the existing brush-type aerators in Oxidation Ditch No. 1 have an oxygenation capacity of 6.6 lbs/hr per foot of aerator shaft length, while the brush aerators in Oxidation Ditch No. 2 have an oxygenation capacity of 5.85 lbs /hr per foot of aerator shaft length. Based on the total shaft length, the combined oxygenation capacity of all aerators is 430 lbs per hour.

The flow and mass balance evaluation for future flow conditions in Appendix A presents the calculated SOR (in pounds per day) for the future annual average, maximum month and maximum daily flows. The resulting SORs are then compared to the existing oxygenation capacity of 430 lbs/hour to assess capacity adequacy.

Based on the comparison of calculated future SOR to existing oxygenation capacity, the existing aerators have sufficient capacity for the future annual average, maximum monthly and maximum daily flows. Therefore, the existing aerators will not require upgrading under any of the future flow conditions.

8.3.2 Tank Volume

The adequacy of oxidation ditch tank volume is evaluated primarily through the following parameters:

- Volumetric BOD loading, which is the pounds of BOD that enter each 1,000 cubic feet (CF) of tank volume per day.
- Hydraulic detention time, which is the time required for each gallon of wastewater to flow through the tank.
- Solids retention time (SRT), which is the time that each pound of biomass resides in the biological treatment system before it is removed from the system as waste sludge. SRT is the key parameter that controls the type and distribution of microorganisms present in the oxidation ditch, and also controls the floc forming characteristics of the microorganisms thereby directly influencing settling characteristics of the biomass. Thus an appropriate SRT must be selected for a flow and mass balance.

As shown in Table 1, the volume of Oxidation Ditch No. 1 is 293,000 gallons and the volume of Oxidation Ditch No. 2 is 614,000 gpd. The resulting total volume is 907,000 gallons.

The flow and mass balances evaluations for future flow in Appendix A present the calculated volumetric BOD loading and hydraulic detention time for the future annual average, maximum month and maximum daily flows and loads. The resulting volumetric BOD loading and hydraulic detention time are compared to standard sizing criteria from a variety of sources including the NJDEP, 10-States Standards, and Water Environment Federation Manual of Practice No. 8 Design of Municipal Wastewater Treatment Plants. Based on a comparison of the calculated volumetric loading and hydraulic detention time versus standard sizing criteria, the oxidation tank volume is sufficient for all future flows.

With regard to SRT, the flow and mass balance evaluations in Appendix A allow an SRT to be selected, and for the spreadsheet to then calculate the resulting MLSS concentration and mass of sludge wasted per day. For the future flows, SRTs have been selected based on kinetic analysis to ensure a high level of BOD and NH_3 removal, as validated through current performance operating at similar SRTs. The resulting MLSS concentrations are typical of MLSS concentrations in oxidation ditches, which also indicate that oxidation ditch volume is sufficient for the future flows. The resulting MLSS concentrations are also used in the assessment of clarifier and return sludge pumping system capacity, as discussed in Section 8.4.

In summary, considering both tank volume and aeration capacity, the existing oxidation ditches are sufficiently sized for future flows.

8.4 Final Clarifiers and Return Sludge Pumping System

Final clarifiers are integral components of the biological treatment system as they allow the biomass (i.e. MLSS) that flows out of the oxidation ditches to be settled and thickened for return to the oxidation ditches (by way of the return sludge pumps). They also produce a clarified effluent low in total suspended solid concentration. Final clarifier and return sludge pumping system capacity must be evaluated together, as the capacity of the return sludge pumping system directly impacts clarifier capacity.

Final clarifier capacity is evaluated based on two key criteria:

1. Surface overflow rate, which is the gallons per day of wastewater flow to the final clarifiers divided by the clarifier surface area.
2. Solids loading rate, which is the pounds per day of biomass (i.e. MLSS) applied to the final clarifiers divided by the clarifier surface area.

Surface overflow rate relates to the clarification function of a clarifier while the solids loading rate relates to the thickening function of a clarifier. Failure of either the clarification or thickening function results in overall failure of the clarifier. Therefore, the more stringent of these two criteria determines the overall capacity of the clarifier.

As indicated in Table 1, there are two final clarifiers, each 50 feet in diameter with a sidewater depth of 11'-8". Settled biomass is returned to the oxidation ditches by the return sludge pumping system, which consists of four variable speed pumps, each rated for 425 gpm at 25 feet TDH. However, the return sludge piping is configured such that two pumps are dedicated to each clarifier. Therefore, the firm capacity of the return sludge pumping system (i.e. with 1 pump serving each final clarifier out of service for maintenance) is approximately 850 gpm or approximately 1.2 mgd.

The adequacy of the clarification and thickening functions of the clarifier are evaluated separately below.

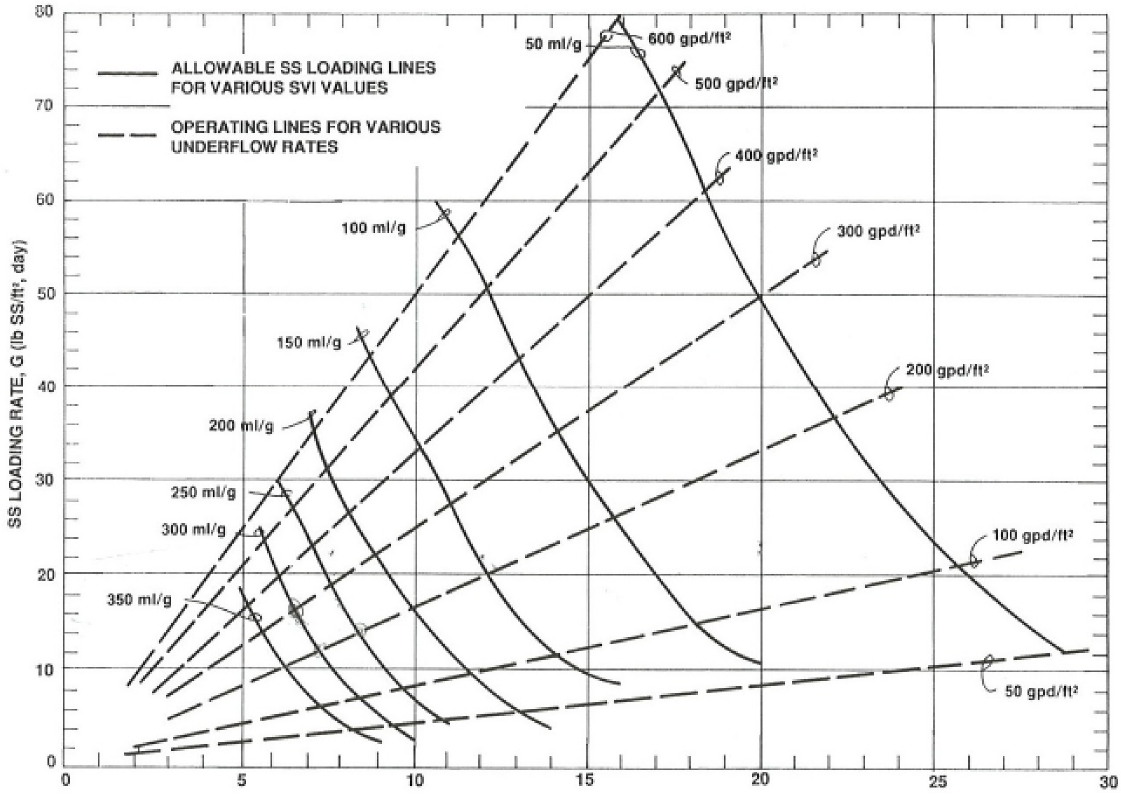
8.4.1 Clarification

For clarification to occur, the upflow velocity (i.e. surface overflow rate) of the clarifier must be less than the settling velocity of a typical biomass particle. The flow and mass balance evaluation for future flows in Appendix A present the calculated surface overflow rate for the future annual average, maximum month, maximum daily and peak hourly flows. The resulting surface overflow rates are compared to recommended surface overflow rates from a variety of sources including the NJDEP, 10-States Standards, and Water Environment Federation Manual of Practice No. 8 Design of Municipal Wastewater Treatment Plants. Based on a comparison of the calculated surface overflow rate versus the recommended surface overflow rates, the final clarifiers have sufficient clarification capacity for the future annual average, maximum month and maximum day flows, and marginally acceptable clarification capacity during the future peak hourly flow, which is acceptable provided the effluent filters have sufficient capacity to accommodate the peak hourly flow..

8.4.2 Thickening

When thickening failure occurs, biomass will be "washed out" of the clarifier resulting in poor effluent quality and potentially long term disruption in system performance. For proper thickening to occur, the solids loading cannot exceed a maximum rate dictated by the settleability (i.e. SVI) of the MLSS and the underflow rate of the clarifier. The underflow rate is the return sludge flow rate divided by the clarifier surface area. This type of analysis is referred to as a State Point Analysis and is performed using the type of diagram presented in Figure 6 below, which is from the *Manual on the Causes and Control of Activated Sludge Bulking and Foaming, 2nd Edition by Jenkins, Richard and Daigger*.

Figure 6: Clarifier Diagram for State Point Analysis



In Figure 6 the dashed lines are the underflow lines, which represent the return sludge flow rate divided by the clarifier surface area, and the vertical lines are the SVI lines. The intersection of an underflow line with an SVI line establishes a maximum allowable solids loading rate above which thickening failure will occur. For example, at an underflow rate of 200 gpd/ft^2 and an SVI of 150 ml/g, the maximum allowable solids loading is 20 pounds of MLSS per day per square feet of clarifier surface area.

This diagram was first used to assess thickening capacity at the existing influent flows, typical RAS flow, typical MLSS concentrations, and typical range of SVI values. As indicated in the flow and mass balance evaluation for existing conditions in Appendix A, at the current typical return sludge flow rate of 0.4 mgd, the resulting underflow rate is approximately 100 gpd/ft^2. At a typical SVI ranging between 100 ml/g and 150 ml/g, and by referring to Figure 6, the corresponding maximum allowable solids loading rate is in the range of 11 to 15 pounds per day per square feet. Based on the calculated solids loading rates presented in the flow and mass balance evaluations for existing conditions in Appendix A, this range of maximum allowable solids loading rates is exceeded, and thus thickening failure is expected to occur, about midway between the current maximum monthly flow of 1.9 mgd and the current maximum daily flow of 3.47 mgd. This flow and mass balance prediction is consistent with reported actual observed conditions, because plant operators must shutoff the oxidation channel aerators as flows approach 3 mgd to settle MLSS in the oxidation ditches to reduce the final clarifier solids loading rate.

The flow and mass balance evaluation for future flows in Appendix A presents the maximum achievable thickening capacity by operating the return sludge pumping system at its firm capacity of 1.2 mgd. As indicated, the final clarifiers have adequate thickening capacity for the future annual average and maximum monthly average flows. However, at the maximum daily flow and a return sludge pumping rate of 1.2 mgd, the thickening capacity is not adequate even at optimal MLSS settling characteristics (i.e. an SVI of less than or equal to 100 ml/g). Therefore, without an increase in return sludge pumping capacity, solids washout and disruption of the biological treatment process would occur during future maximum daily flow conditions.

Therefore, to accommodate the full range of future flows, it is recommended that the firm capacity of the return sludge pumping system be increased from 1.2 mgd to 2.5 mgd, which would result in adequate thickening capacity during future maximum daily flows at SVIs between 100 ml/g and 150 ml/g.

8.5 Waste Sludge Pumping System

Waste sludge pumps are used to remove biomass from the biological treatment system as required to maintain the desired SRT, which as previously indicated is the key process control parameter related to performance.

The existing waste sludge pumping system consists of two variable speed pumps each rated for 470 gpm (0.68 mgd) at 12 feet TDH. The firm capacity of the pumping system, i.e. with one pump out of service, is 470 gpm or 0.68 mgd.

The flow and mass balance evaluation for future flows in Appendix A present the calculated mass of waste sludge based on the selected SRT, the concentration of waste sludge based on the return sludge flow rate, and the resulting average daily sludge flow in gpd based on the mass and concentration of waste sludge. The maximum waste sludge pumping rate is also presented based on the scenario of pumping 7 days of waste sludge over a 4 day period, at 4 hours per day.

Based on a comparison of the calculated maximum pumping rates presented in the flow and mass balances versus the firm capacity of the waste sludge pumping system, the existing waste sludge pumping system has sufficient capacity for all future flow scenarios.

The flow and mass balances also indicate, as expected, that operating at an increase return sludge flow rate to increase clarifier thickening capacity results in a thinner waste sludge and thus an increase in the volume of sludge that must be mechanically thickened.

8.6 Effluent Filters

Effluent filters remove additional suspended solids that are not removed in the final clarifiers, and thereby remove additional effluent particulate BOD associated with the effluent total suspended solids (every 1 mg/L of effluent TSS typically corresponds to an effluent particulate BOD of 0.6 mg/L. Effluent filters are sized based recommended filtration rates during average and peak flow conditions. In addition, the influent TSS concentrations must be below the manufacturer's recommended maximum values.

The existing effluent filters consist of four Dynasand continuous backwash, upflow, deep bed, single media filters, manufactured by Parkson Corporation. Each filter is 11'-8" in height and has inside filter dimensions of 10' wide by 15' long. The filtration area of each filter is 150 square feet for a total filtration area of 600 square feet.

The flow and mass balance evaluations for future flows in Appendix A present the calculated filtration rate for the annual average, maximum monthly, maximum daily and peak hourly flows. The resulting filtration rates are compared to recommended filtration rates from a variety of sources including the manufacturer, the 10-States Standards, and the M&E Wastewater Engineering Textbook. However, WWTP staff indicate that in actual practice, the existing filters can only handle a maximum flow of 2.8 mgd, and that peak flows in excess of 2.8 mgd must be bypassed around the effluent filters.

Therefore, the existing filters have sufficient capacity for the future annual average and maximum monthly average flows, but do not have adequate capacity for future maximum daily flows or future peak hourly flows. Accordingly, to accommodate the full range of future flows, it is recommended that the peak hourly flow capacity of the effluent filters be increased from 2.8 mgd to 5.2 mgd.

8.7 Post Aeration System

The post aeration system provides additional oxygenation of the filtered effluent so that the NJPDES permit requirement for minimum dissolved oxygen concentration (6 mg/L) can be met. The post aeration system consists of a post aeration tank and a coarse bubble diffuser system that receives air from two blowers, each with a capacity of 220 cfm. Therefore, the firm capacity of the blower system is 220 cfm.

The flow and mass balance for future flows in Appendix A calculates the required blower capacity based on the continued use of coarse bubble diffusers. Based on a comparison of the calculated blower capacity versus the firm capacity of the existing blowers, the blowers/coarse bubble diffusers have sufficient capacity for the future annual average, maximum monthly average, maximum daily average and peak hourly flows.

8.8 UV Disinfection System

The UV disinfection system disinfects the final effluent prior to discharge so the NJDPES permit effluent limitation for fecal coliform organisms can be achieved. UV disinfection systems are designed to deliver a sufficient UV dose to disinfect the wastewater during annual average, maximum monthly average and maximum daily flow conditions. The NJDEP requires that a 125% safety factor be incorporated into the design of the system without referencing a specific flow condition.

The existing UV disinfection system was manufactured by Trojan Technologies and is a horizontal, open channel configuration. The manufacturer's stated maximum daily flow capacity of the system is 4.4 mgd.

The flow and mass balance evaluation for future flow condition in Appendix A present the actual safety factor for the future annual average and maximum monthly flows.

Based on a comparison of the calculated safety factors versus the NJDEP required safety factor, the existing UV disinfection system provides a safety factor significantly in excess of 125% for annual average and maximum monthly average flows. Therefore, this system does not require modification to accommodate future flows.

8.9 Sludge Storage

There are two aerated sludge storage tanks, each with a useable volume of approximately 75,000 gallons. Sludge Storage Tank No. 2 receives waste activated sludge by way of the waste activated sludge pumps. Sludge Storage Tank No. 1 receives thickened waste activated sludge from a mechanical thickener located in the Digester Control Building.

As shown in the flow and mass balance evaluation in Appendix A, the increase in mass of sludge produced at future flows is only about 10% greater than the current mass of sludge produced.

The increase in volume of raw waste sludge will depend on the required return sludge flow rate, since this directly impacts the concentration of the raw waste activated sludge.

Assuming the rate and frequency of thickening increases proportional to the increase in raw waste sludge flow rate, the existing sludge storage tanks have sufficient capacity for future flows.

8.10 Summary of Capacity Deficiencies for Future Flow Scenarios

The plant components with insufficient capacity for future flows are listed below.

4. Influent Pumps
5. Return Sludge Pumps
6. Effluent Filters

Alternatives to address capacity deficiencies are presented in Section 9.0.

9.0 SYSTEM IMPROVEMENT ALTERNATIVES FOR FUTURE FLOW

Based on the capacity deficiencies listed in Section 8.10, there are two basic alternatives for WWTP improvements that will enable future development within the sewer service area:

3. Construct a flow equalization system to temporarily store peak wet weather flow in excess of WWTP capacity.
4. Increase the peak flow capacity of each capacity deficient plant component listed in Section 8.10.

The specific capital improvements associated with each alternative are presented in the sections that follow.

9.1 Flow Equalization Alternative

Under this alternative, the following improvements would be required.

- New influent flow equalization tank (including mixing system and comminutor to grind debris entering the tank.
- Influent pumping system upgrade (replace pumps 3 and 4) to increase firm capacity from 3.4 mgd to 5.2 mgd.

In the original Capacity Assurance Report, the required equalization tank storage volume was estimated to be 1.65 million gallons. Because there has been no quantifiable reduction in I&I flow resulting from the sewer rehabilitation system project, the originally estimated volume remains valid. The lowest cost option for constructing a tank of this volume is to utilize a wire-wound, pre-stressed concrete storage tank. Tank dimensions would be approximately 90' diameter with a 35' side water depth. A budgetary capital cost estimates for WWTP improvements was developed using the same methodology as in the original Capacity Assurance Report. The resulting budgetary capital cost estimate in 2018 dollars is presented in Table 11.

Table 11: Budgetary Capital Cost Estimate for Flow Equalization Alternative

Item/Description	*Quantity*	*Unit/Basis*	*Unit Budgetary Cost*	*Item Budgetary Cost*
Plant Improvement Budgetary Costs				
Major Equipment & Systems				
Flow Equalization Tank (installed)	1	LS	$ 1,400,000	$ 1,400,000
Comminutor	1	LS	$ 25,000	$ 25,000
Jet Mix System	1	LS	$ 186,000	$ 186,000
Replace Influent Pump 3 and 4	2	LS	$ 45,000	$ 90,000
Subtotal				*$ 1,701,000*
Installation (excluding EQ Tank)	25%			$ 75,250
Major Equipment and Systems Subtotal				*$ 1,776,250*
Foundations				
Flow Eq Tank Foundation (Piles)	6,500	SF	$ 27.00	$ 175,500
Buildings/Foundations Subtotal				*$ 175,500*
Percentage Items				
Civil/Site	5%			$ 97,588
Piping	5%			$ 97,588
Electrical	15%			$ 292,763
Instrumentation & Controls	5%			$ 97,588
Contractor OH&P	21%			$ 532,828
Contingency	25%			$ 634,319
Percentage Items Subtotal				*$ 1,752,672*
Plant Improvements Budgetary Construction Cost				***$ 3,704,422***
Design and Construction Phase Engineering	18%			$ 666,796
TOTAL BUDGETARY CAPITAL COST				**$ 4,371,217**

A conceptual flow schematic showing how the equalization tank would be incorporated into the plant is presented as Figure 7. A conceptual site plan showing a possible location for the tank is presented as Figure 8.

9.2 Plant Expansion

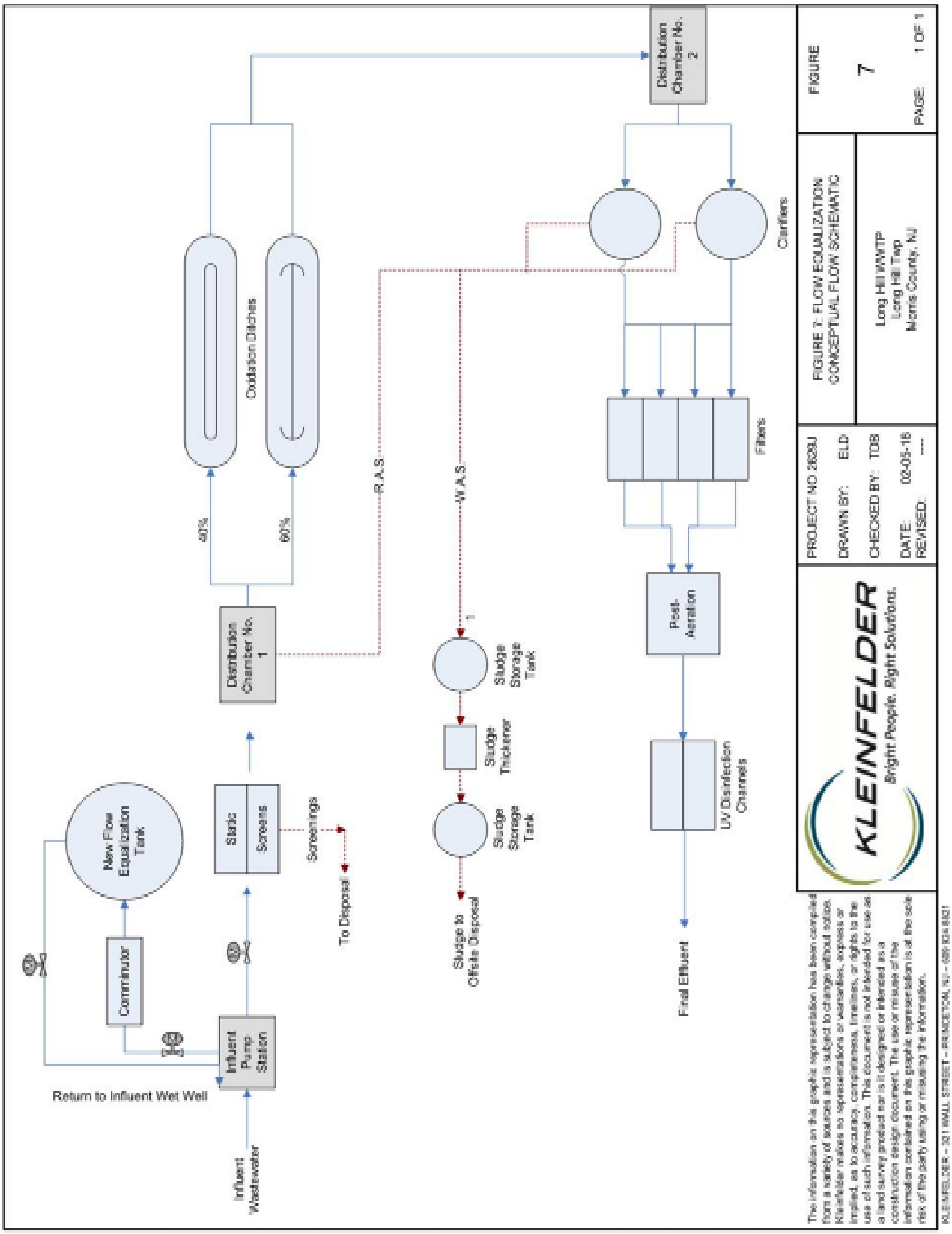
Under this alternative, the following improvements would be required.

- Influent pumping system upgrade (replace pumps 3 and 4) to increase firm capacity from 3.4 mgd to 5.2 mgd.
- Return sludge pumping system upgrade (replace all four pumps) to increase firm capacity from 1.2 mgd to 2.5 mgd.
- Replace existing sand filters with new compact filters to increase peak flow capacity from 2.8 mgd to 5.2 mgd.

Because the existing sand filters are located within an existing building in which there is no available space to construct additional sand filters, the design concept to avoid construction of a new Filter Building is to replace the existing sand filters with a compact type of filter equipment referred to as disc filters. In addition to the significant advantage of being a very compact technology, disc-type effluent filters have additional advantages including low cost, low headloss and low backwash volume.

While several configurations of disc filters exist, the “inside-out” flow path has been assumed. With this flow path, final clarifier effluent enters the filter through a center feed tube with slots or ports to distribute the flow to the discs. Solids are retained on the inside of the discs as the flow passes through the disc filter mesh, and the filtered effluent passes to the outside of the discs and into the collection tank and effluent outlet. As solids are captured, the liquid level within the feed zone rises until it reaches a pre-set level. A sensor then initiates operation of a wash water pump which pumps the filtered effluent through spray nozzles, removing the accumulated solids from the discs. The solids and wash water are collected in a trough and conveyed by the plant drain to the head of the plant for treatment. The back wash water volume is typically about 1% of the total flow to the filter.

Figure 9 on Page 33 shows the general arrangement of this type of disc filter in self-contained tanks.



Influent Wastewater
Return to Influent Wet Well
Influent Pump Station
Comminutor
New Flow Equalization Tank
Static Screens
Screenings
To Disposal
Distribution Chamber No. 1
40%
60%
Oxidation Ditches
Distribution Chamber No. 2
R.A.S.
W.A.S.
Sludge Storage Tank
Sludge Thickener
Sludge Storage Tank
Sludge to Offsite Disposal
Clarifiers
Filters
Post-Aeration
UV Disinfection Channels
Final Effluent
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FIGURE 7: FLOW EQUALIZATION CONCEPTUAL FLOW SCHEMATIC
Long Hill WWTP
Long Hill Twp
Morris County, NJ
FIGURE
7
PAGE: 1 OF 1

FIGURE 8:
FLOW EQUALIZATION CONCEPTUAL SITE PLAN

Long Hill WWTP
Long Hill Township
Morris County, NJ

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Streams
Conceptual Improvements
Property Boundary

Sources:
2002 NHD Streams
2011 NJDEP Surface Water Quality Streams
2018 ESRI Basemaps

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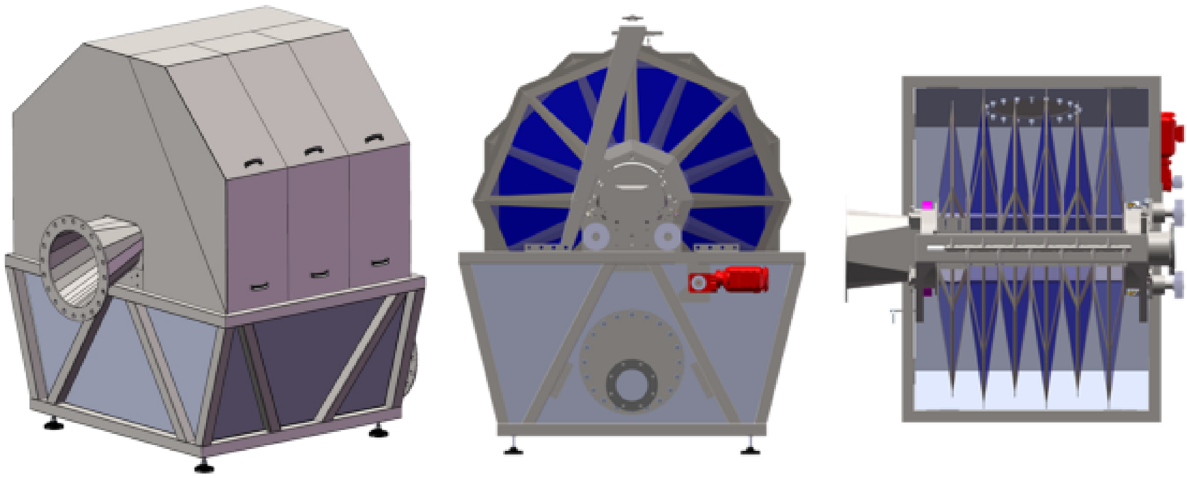

Figure 9: Disc Filter General Arrangement

This type of disc filter is fabricated from stainless steel and will capture all particles larger than 10 to 20 micron, resulting in an effluent TSS concentration less than 5 mg/L. A three-filter installation is recommended with two operating and one standby, each sized for 5% of the future peak hourly flow. The three filters will occupy a smaller space than currently occupied by the four existing sand filters.

The budgetary capital cost estimate in 2018 dollars for the Plant Expansion Alternative is presented in Table 12 on the following page.

Therefore, the $2.8 million budgetary capital cost for the Plant Expansion Alternative is $1.6 million less than the $4.4 million budgetary capital cost for the Flow Equalization Alternative.

9.3 Environmental Constraints

All of the capacity improvements described above, except the new flow equalization tank, would be installed within existing structures or buildings. To assess potential regulatory challenges related to construction of a flow equalization tank, the existing WWTP site was evaluated for environmental constraints including threatened and endangered species habitat, riparian zones, wetlands, and flood hazard area, which are presented in Figures 10, 11, 12 and 13, respectively, beginning on Page 35.

Table 12: Budgetary Capital Cost Estimate for Plant Expansion Alternative

Item/Description	*Quantity*	*Unit/Basis*	*Unit Budgetary Cost*	*Item Budgetary Cost*
Plant Improvement Budgetary Costs				
Major Equipment & Systems				
Disc Filters	3	LS	$ 160,000	$ 480,000
New Influent Pump #3 and #4	2	LS	$ 45,000	$ 90,000
New RAS Pumps with VFDs	4	LS	$ 55,000	$ 220,000
Valves, and Interior Piping	1	LS	$ 100,000	$ 100,000
Platforms and Handrail	1	LS	$ 40,000	$ 40,000
Subtotal				*$ 930,000*
Installation	25%			$ 232,500
Major Equipment and Systems Subtotal				*$ 1,162,500*
Demolition and Foundations				
Demolition of Existing Filters	1	LS	$ 40,000	$ 40,000
Disc Filter Foundations	4	LS	$ 8,000	$ 32,000
Buildings/Foundations Subtotal				*$ 72,000*
Percentage Items				
Civil/Site	5%			$ 61,725
Piping	5%			$ 61,725
Electrical	15%			$ 185,175
Instrumentation & Controls	5%			$ 61,725
Contractor OH&P	21%			$ 337,019
Contingency	25%			$ 401,213
Percentage Items Subtotal				*$ 1,108,581*
Plant Improvements Budgetary Construction Cost				***$ 2,343,081***
Design and Construction Phase Engineering	18%			$ 421,755
TOTAL BUDGETARY CAPITAL COST				**$ 2,764,836**

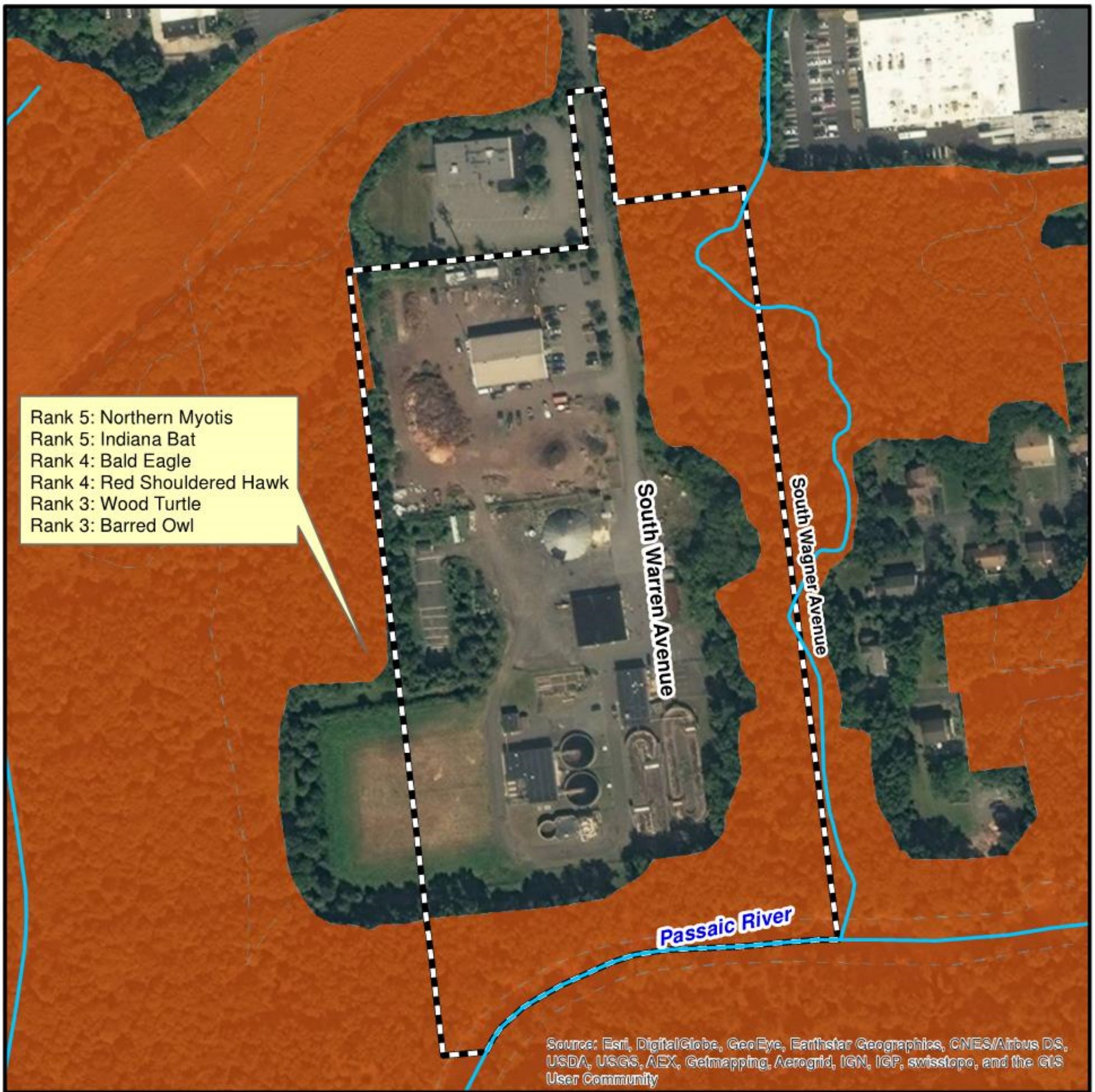

FIGURE 10: THREATENED & ENDANGERED SPECIES MAP

Long Hill WWTP
Long Hill Township
Morris County, NJ

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FIGURE 11:
RIPARIAN ZONE MAP

Long Hill WWTP
Long Hill Township
Morris County, NJ

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Streams
Property Boundary
Approx. 150' Riparian Zone

Sources:
2002 NHD Streams
2011 NJDEP Surface Water Quality Streams
2018 ESRI Basemaps

Note: The Riparian Zone is estimated to be 150' based on stream quality classification and presence of endangered species critically dependent on water for survival. Top of Bank was esitmated based on aerial imagery.

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FIGURE 12:
WETLANDS MAP

Long Hill WWTP
Long Hill Township
Morris County, NJ

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Streams
Property Boundary
Approx. 150' Wetlands Transition Area
Wetlands (NJDEP 2012)

Sources:
2002 NHD Streams
2011 NJDEP Surface Water Quality Streams
2012 NJDEP Land Use/ Land Cover
2018 ESRI Basemaps

Note: The Wetland Transition area was assumed to be 150' due to presence of threatened and endangered species.

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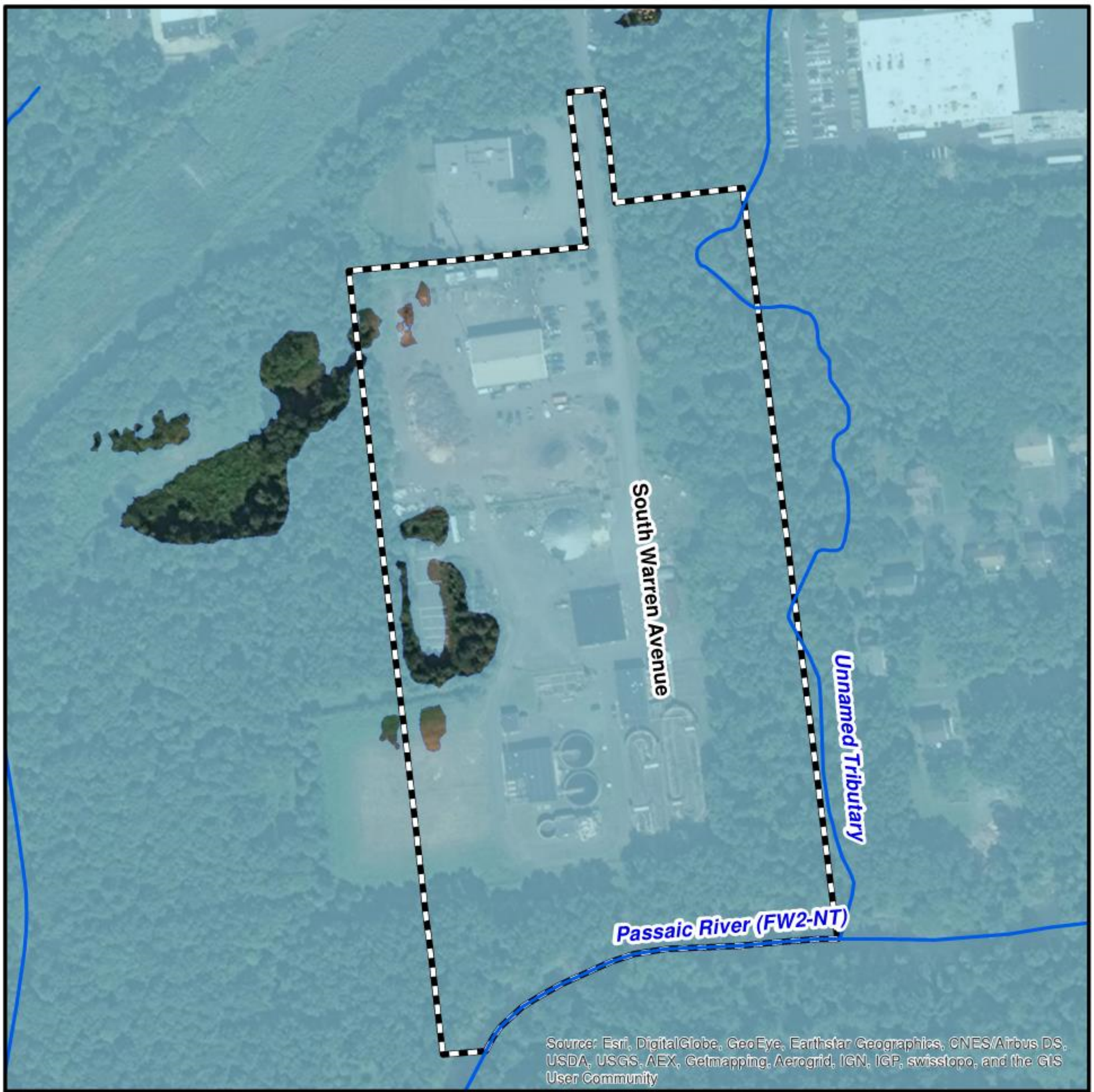

FIGURE 13: FLOOD HAZARD AREA MAP

Long Hill WWTP
Long Hill Township
Morris County, NJ

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Streams
Property Boundary
Flood Hazard Area

Sources:
2002 NHD Streams
2011 NJDEP Surface Water Quality Streams
2018 ESRI Basemaps
2007 USGS LiDAR Topography

Note: The FHA Elevation was determined to be 215.09 (NAVD88) based upon a review of the NJDEP Studies and Available FEMA mapping. Location of the FHA was estimated based on LiDAR topography.

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Based on Figures 10, 11 and 12, threatened and endangered species habitat, riparian zones and wetlands are all beyond the area of the site that would be impacted by construction of the flow equalization tank. Therefore, these environmental constraints would not cause significant regulatory challenges in constructing a flow equalization tank.

However, Figure 13 indicates that the entire WWTP site is within the NJDEP Flood Hazard Area. Therefore, a flood hazard area permit would be required for construction of a flow equalization tank. A key requirement for obtaining a flood hazard area permit is that the flood storage volume displaced by the new structure will need to be fully offset in another location so that there is no net decrease in flood storage volume.

10.0 PLANT IMPROVEMENTS FOR FUTURE TP EFFLUENT LIMITS

As described in Section 3.0, a monthly average TP limit of 0.76 mg/L will be imposed in the near future when the WWTP's NJPDES Permit is renewed. As described in Section 6.0, a 0.76 mg/L limit is significantly more stringent than the existing TP effluent limitation, which will necessitate that capital improvements be implemented.

As described in the original Capacity Assurance Report, phosphorus can be removed biologically or chemically. However, biological phosphorus removal (BPR) alone cannot reliably achieve an effluent limitation of 0.76 mg/L. While a portion of the phosphorus can be removed through BPR and the balance through chemical phosphorus removal, the existing oxidation ditches do not have sufficient volume for operation in a BPR mode. Therefore, chemical phosphorus removal alone will be required.

To accomplish chemical phosphorus removal, a coagulant is added to the wastewater which causes soluble phosphorus (orthophosphate) to precipitate and to be removed from the wastewater as a solid. This can be achieved through a variety of add-on processes, such as the CoMag® process, Blue PRO® reactive filter, and Actiflo® micro-sand enhanced clarification process. However, to achieve reliable compliance with a 0.76 mg/L effluent limitation, none of these expensive add-on processes are required; rather, chemical addition in Division Box B upstream of the final clarifiers will be sufficient.

The optimum coagulant should be determined through a site-specific evaluation of alternatives. The evaluation should include jar testing and related analysis based on a comprehensive list of criteria, including performance, chemical cost, sludge production, alkalinity consumption, and increase in TDS concentration. Full scale demonstration testing is also normally recommended to verify design and operational parameters estimated through jar testing.

For the purposes of this Capacity Assurance Report Update, it will be assumed that Polyaluminun Chloride (PACL) will be utilized as the coagulant. PACL offers a number of advantages over alum or ferric chloride, including reduced alkalinity consumption and a reduced increase in sludge production.

Based on recent experience at a nearby authority, it is estimated that the average feed rate of PACL will be approximately 80 gallons per day (this feed rate could be refined for LHT through site-specific bench scale testing). At a current bulk cost of $5.00 per gallon, the corresponding annual chemical cost would be about $146,000 per year. Based on a typical storage volume of 30 days, and pending testing to confirm dose, two 1,500 gallon storage tanks are anticipated, together with chemical feed pumps, which will need to be located in a building (for freeze protection) with spill containment. To minimize cost, a pre-engineered metal building will be assumed.

The addition of a coagulant will also increase sludge production, typically by about 20%, resulting in approximately a 20% increase in current sludge disposal costs.

Budgetary costs for the phosphorus removal improvements were developed based on budgetary costs from a representative chemical storage tank manufacturer (Snyder), budgetary costs from typical unit costs for a pre-engineered metal building, and percentages for items such as equipment installation, site work, piping, equipment installation and electrical, together with contractor overhead and profit at 21% (includes mobilization bonds and insurance), a 25% contingency, and 18% for design and construction phase engineering services. The resulting budgetary capital cost estimate, in 2018 dollars, is presented in Table 13.

A conceptual flow schematic and site plan are presented as Figures 14 and 15.

Table 13: Budgetary Capital Cost Estimate for Future TP Effluent Limitation

Item/Description	*Quantity*	*Unit/Basis*	*Unit Budgetary Cost*	*Item Budgetary Cost*
Plant Improvement Budgetary Costs				
Major Equipment & Systems				
PACL Storage Tanks	2	EA	$ 4,000	$ 8,000
PACL Feed System	1	LS	$ 25,000	$ 25,000
Spill Containment	1	LS	$ 30,000	$ 30,000
Platform and Handrail	1	EA	$ 20,000	$ 20,000
				$ -
Subtotal				*$ 83,000*
Installation	25%			$ 20,750
Major Equipment and Systems Subtotal				*$ 103,750*
Building				
Pre-Engineered Building (including HVAC)	900	SF	$ 180.00	$ 162,000
Concrete Foundation	100	CY	$ 600.00	$ 60,000
Building Subtotal				*$ 222,000*
Percentage Items				
Civil/Site	10%			$ 32,575
Piping and Valves	10%			$ 32,575
Electrical	12%			$ 39,090
Instrumentation & Controls	5%			$ 16,288
Contractor Overhead & Profit	21%			$ 93,718
Contingency	25%			$ 111,569
Percentage Items Subtotal				*$ 325,815*
Plant Improvements Budgetary Construction Cost				***$ 651,565***
Design and Construction Phase Engineering	18%			$ 117,282
TOTAL BUDGETARY CAPITAL COST				**$ 768,847**

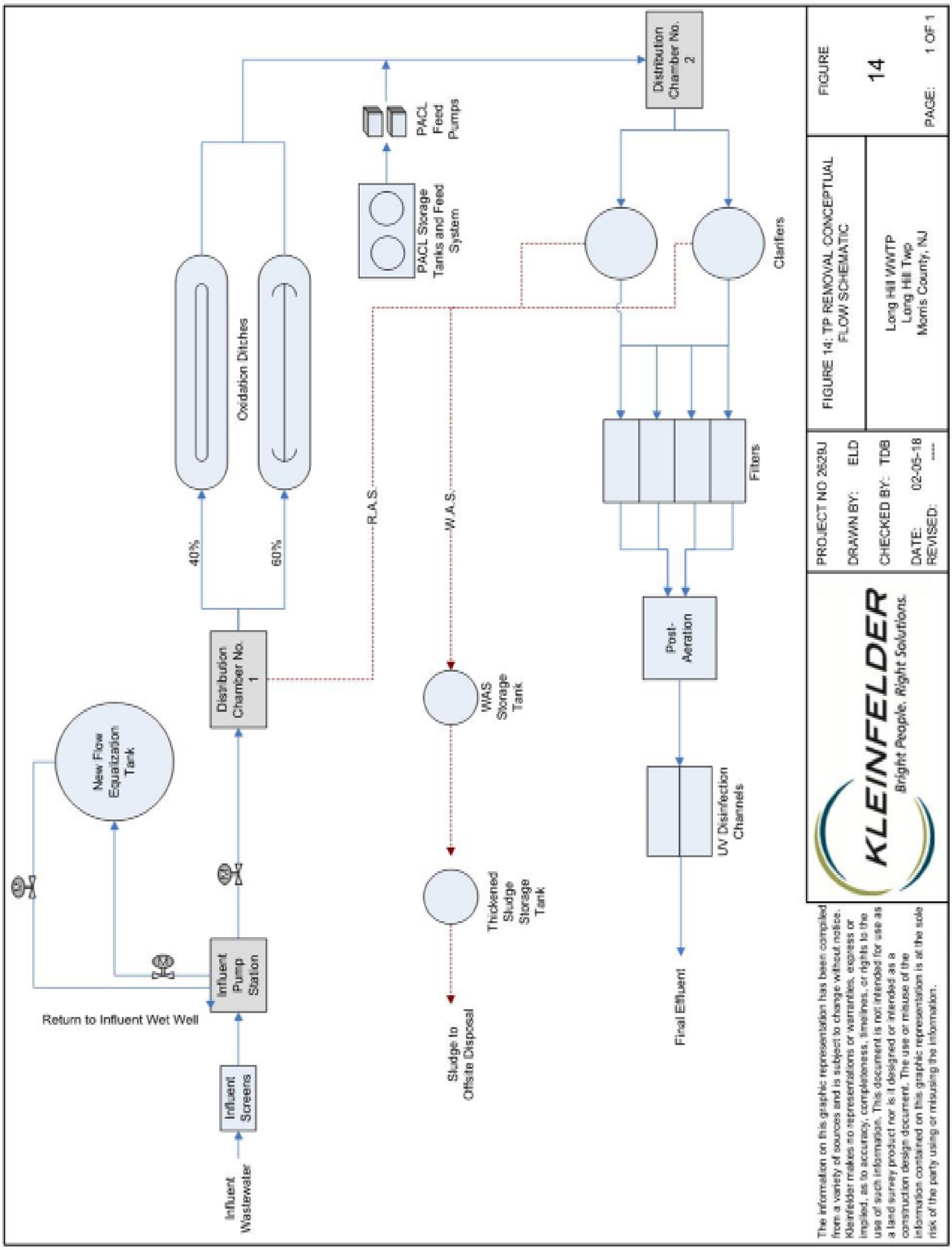
Influent Wastewater
Influent Screens
Influent Pump Station
Return to Influent Wet Well
New Flow Equalization Tank
Distribution Chamber No. 1
40%
60%
Oxidation Ditches
R.A.S.
W.A.S.
PACL Storage Tanks and Feed System
PACL Feed Pumps
Distribution Chamber No. 2
Clarifiers
Filters
Post-Aeration
UV Disinfection Channels
Final Effluent
WAS Storage Tank
Thickened Sludge Storage Tank
Sludge to Offsite Disposal
PROJECT NO 2629J
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FIGURE 14: TP REMOVAL CONCEPTUAL FLOW SCHEMATIC
Long Hill WWTP
Long Hill Twp
Morris County, NJ
FIGURE
14
PAGE: 1 OF 1
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**FIGURE 15:
TP REMOVAL
CONCEPTUAL SITE PLAN**

Long Hill WWTP
Long Hill Township
Morris County, NJ

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Streams
Conceptual Improvements
Property Boundary

Sources:
2002 NHD Streams
2011 NJDEP Surface Water Quality Streams
2018 ESRI Basemaps

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11.0 CONCLUSIONS AND RECOMMENDATIONS

The key conclusions and recommendations resulting from the Capacity Assurance Report Update are:

1. There has not been a measurable reduction in sanitary sewer system I&I resulting from the recent sewer rehabilitation project encompassing 4.6% of the overall system.
2. The lowest cost and recommended alternative to provide sufficient capacity for future growth is the plant expansion alternative.
3. The budgetary capital cost estimate for the plant expansion alternative is approximately $2.8 million in 2018 dollars, based on implementation of the following plant improvements:
    a. Replacement of the four (4) existing sand filters with three (3) disc filters to increase peak hour flow capacity of the effluent filters from 2.8 mgd to 5.2 mgd.
    b. Replacement of Influent Pumps #3 and #4 with larger units to increase the firm capacity of the influent pumping system from 3.4 mgd to 5.2.
    c. Replacement of the four (4) existing return sludge pumps with larger units to increase the firm capacity of the return sludge pumping system from 1.2 mgd to 2.5 mgd.
4. NJDEP will lower the TP effluent limit to 0.76 mg/L when it renews the NJPDES Permit, likely within the next few months. The budgetary capital cost estimate for the improvements needed to achieve compliance with the new TP effluent limit is approximately $0.8 million. Pending site-specific testing to confirm actual coagulant dose, the estimated annual chemical cost to achieve compliance with this limit is approximately $146,000 per year. Coagulant addition will increase sludge generation, typically by approximately 20%, resulting in approximately a 20% increase in the current annual cost of sludge disposal.
5. Without ongoing I&I reduction activities, the low rate of I&I will increase in the future as the wastewater collection system continues to age and deteriorate. Therefore, it is recommended that sewer rehabilitation improvements be implemented on a regular basis, similar to the regular implementation of road re-paving projects.
6. The WWTP's NJPDES Permit will need to be modified to increase the permitted capacity from 0.9 to 1.24 mgd. Because NJPDES Permits must be consistent with the relevant WMP, the LHT WMP will need to be amended before the modified NJPDES Permit will be approved by NJDEP.

APPENDIX A

Flow and Mass Balance Evaluations – Existing and Future Flow

LONG HILL TOWNSHIP WWTP
FLOW AND MASS BALANCE EVALUATION
EXISTING FLOW CONDITIONS

	UNITS	ANNUAL AVERAGE DAY	MAXIMUM MONTH	MAXIMUM DAY	PEAK HOURLY FLOW
INFLUENT FLOWS & LOADS					
Influent Flow	mgd	1.04	1.91	3.47	4.40
Influent TSS Concentration	mg/l	117	113	198	
Influent TSS Load	lbs/day	1,016	1,805	5,744	
Influent BOD Concentration	mg/l	99	87	121	
Influent BOD Load	lbs/day	859	1,381	3,517	
Influent TKN Concentration	mg/l	18	16	22	
Influent TKN Load	lbs/day	156	251	639	
INFLUENT + RECYCLE FLOWS & LOADS					
Influent Flow	mgd	1.09	2.01	3.58	4.54
Influent TSS Concentration	mg/l	120	116	207	
Influent TSS Load	lbs/day	1094	1942	6167	
Influent BOD Concentration	mg/l	99	87	126	
Influent BOD Load	lbs/day	906	1463	3771	
Influent TKN Concentration	mg/l	17	15	21	
Influent TKN Load	lbs/day	156	253	642	
OXIDATION CHANNELS					
Influent Flow	mgd	1.09	2.01	3.58	4.54
Influent TSS	lbs/day	1094	1942	6167	
Influent TSS	mg/l	120	116	207	
Influent BOD	lbs/day	906	1463	3771	
Influent BOD	mg/l	99	87	126	
Influent TKN	lbs/day	156	253	642	
Influent TKN	mg/l	17	15	21	
Total Volume Installed	ft^3	121,249	121,249	121,249	121,249
Total Volume Installed	gals	907,000	907,000	907,000	907,001
Total Volume in Service	ft^3	121,249	121,249	121,249	121,249
Total Volume in Service	gals	907,000	907,000	907,000	907,001
Typical Design Criteria					
Hydraulic D.T. @ Design Flow (NJDEP)	hrs	7.5	7.5	N/A	N/A
Hydraulic D.T. @ Design Flow (WEF)	hrs	8-36	8-36	N/A	N/A
BOD Loading @ Design Flow(NJDEP)	lb/Kcf/day	38	38	N/A	N/A
BOD Loading @ Design Flow(10 STATES)	lb/Kcf/day	15	15	N/A	N/A
BOD Loading @ Design Flow(WEF)	lb/Kcf/day	5-30	5-30	N/A	N/A
Solids Retention Time @ Design Flow (WEF)	days	10-30	10-30	N/A	N/A
F/MLVSS Ratio at Desgin Flow (M&E Wastewater Engineerin Text)		0.04-0.1	0.04- 0.1	N/A	N/A
Actual Hydraulic Detention Time	hrs	19.9	10.8	6.1	4.80
Actual BOD Loading	lb/Kcf/day	7	12	31	
Solids Retention Time	days	24	18	10	
MLSS	mg/l	2,073	2,535	3,643	
MLSS	lbs	15,678	19,176	27,559	
Percent MLVSS	%	75%	75%	75%	
MLVSS	lbs	11,758	14,382	20,670	
F/MLSS		0.06	0.08	0.14	
F/MLVSS		0.08	0.10	0.18	
Sludge Production/lb BOD Removed	lb/lb	0.70	0.70	0.70	
Biological Waste Sludge Production	lbs/day	589	928	2,368	
OXYGEN REQUIREMENTS					
Oxygen Required/BOD	lb/lb	1.3	1.3	1.3	
Influent TKN	mg/l	17	15	21	
Influent TKN	lbs/day	156	253	642	
Effluent NH3	mg/l	0.41	2.00	3.00	
Effluent NH3	lbs/day	4	34	90	
Oxygen Required/TKN	lbs/day	4.57	4.57	4.57	
Carbonaceous Oxygen Demand	lbs/day	1,094	1,723	4,398	
Nitrogenous Oxygen Demand	lbs/day	697	1,001	2,523	
Actual Oxygen Requirement (AOR)	lbs/day	1,791	2,725	6,921	
Operating DO	mg/l	2.0	2.0	1.0	
Water Temperature	°C	25	25	25	
Saturation DO	mg/l	8.02	8.02	8.02	
alpha		0.80	0.80	0.80	
beta		0.98	0.98	0.98	
Standard Oxygen Requirement (SOR)	lbs/day	3,082	4,687	10,172	
Standard Oxygen Requirement (SOR)	lbs/hr	128	195	424	
Oxidation Ditch #1 Brush Aerator Capacity @ 6.6 lb//hr/ft of shaft	lbs/hr	185	185	185	
Oxidation Ditch #2 Brush Aerator Capacity @ 5.85 lb/hr/ft of shaft	lbs/hr	245	245	245	
Total Oxidaiton Ditch Brush Aerator Capactiy	lbs/hr	430.00	430.00	430.00	

LONG HILL TOWNSHIP WWTP
FLOW AND MASS BALANCE EVALUATION
EXISTING FLOW CONDITIONS

	UNITS	ANNUAL AVERAGE DAY	MAXIMUM MONTH	MAXIMUM DAY	PEAK HOURLY FLOW
FINAL CLARIFIERS					
Number of Tanks Installed		2	2	2	2
Number of Tank in Service		2	2	2	2
Clarifier Diameter	ft	50	50	50	50
Clarifer Depth	ft	11.66	11.66	11.66	11.66
Area per Clarifier	sf	1,963	1,963	1,963	1,963
Total Area in Service	sf	3,927	3,927	3,927	3,927
Volume per Clarifier	cf	22,894	22,894	22,894	22,894
Total Volume in Service	cf	45,789	45,789	45,789	45,789
Volume per Clarifier	gals	171,273	171,273	171,273	171,273
Total Volume in Service	gals	342,545	342,545	342,545	342,545
Design Criteria:					
Overflow Rate @ Design Flow (NJDEP)	gpd/sf	<1000	<1000	N/A	N/A
Overflow Rate @ Peak Flow (10 STATES)	gpd/sf	N/A	N/A	<1,000	<1,000
Overflow Rate @ Avg & Peak (WEF)	gpd/sf	400-700	400-700	1000-1600	1000-1600
Solids Loading Rate (10 STATES)	lbs/sf day	N/A	N/A	<35	N/A
Solids Loading Rate (WEF) *Solids Flux Analysis	lbs/sf day	SF*	SF*	SF	
Solids Loading Rate (M&E Wastewater Engineering Text)	lbs/sf day	12-24	12-24	<34	N/A
RAS Flow % of Influent Flow (M&E Wastewater Engineering Text)	%	75 to 150	75 to 150	N/A	N/A
Solids Flux Capacity at RAS flow and SVI between 100 and 150	lbs/sf day	11 to 15	11 to 15	11 to 15	
Actual Overflow Rate	gpd/sf	279	512	911	1,155
Actual Solids Loading	lbs/sf day	6	12	30	
RAS Pumping System Capacity	mgd	1.2	1.2	1.2	
RAS Flow	mgd	0.40	0.40	0.40	
Underflow Rate (RAS flow divided by clarifer surface area)	gpd/sf	101.86	101.86	101.86	
RAS % of Influent Flow	%	37%	20%	11%	
RASS	mg/l	7,417	14,695	34,379	
RASS	lbs/day	24,447	48,435	113,313	
MLSS	mg/l	2,073	2,535	3,643	
Total Flow (Plant + RAS)	mgd	1.44	2.31	3.87	
MLSS Load	lbs/day	24,891	48,838	117,591	
WAS Production	lbs/day	589	928	2,368	
WAS Solids Content	mg/l	7,417	14,695	34,379	
WAS Solids Content	%	0.74%	1.47%	3.44%	
WAS Flow	mgd	0.010	0.008	0.008	
WAS Flow	gpd	9,527	7,571	8,259	
Clarifier Effluent Flow	mgd	1.09	2.01	3.58	4.54
Clarifier Effluent TSS	mg/l	10.00	12.00	20.00	20
Clarifier Effluent TSS	lb/day	91.30	201.21	596.70	
Clarifier Effluent TSS	kg/day	41	91	271	
Clarifier Effluent BOD	mg/l	7.00	8.20	13.00	
Clarifier Effluent BOD	lb/day	64	137	388	
Clarifier Effluent BOD	kg/day	29	62	176	
Clarifier Effluent NH3	mg/l	0.41	2.00	3.00	
Clarifier Effluent NH3	lb/day	4	34	90	
Clarifier Effluent NH3	kg/day	2	15	41	
FILTERS					
Number of Continuous Backwash Filters		4	4	4	4
Total surface area per filter	SF	150	150	150	150
Total Filter Surface Area	SF	600	600	600	600
Design Criteria					
Filtration Rate (10 States)	gpm/sf	N/A	N/A	N/A	<5
Filtration Rate (M&E Wastewater Engineering Test)	gpm/sf	2	2	<5	<5
Filtration Rate (Manufacturer)	gpm/sf	2 to 3	2 to 3	<5	<5
Maximum TSS concentration (Manufacturer)	mg/L	20 to 30	20 to 30	20 to 30	20 to 30
Actual Filtration Rate	gpm/sf	1.27	2.33	4.14	5.25
Acutal Flter Peak Floow Capacity 2.8 mgd = 3,2 gpm/sf					
Recycle Flow:					
Backwash flow % of Forward Flow	%	5	5	3	3
Backwash flow	mgd	0.05	0.10	0.11	0.14
Backwash TSS	mg/l	171	163	472	
Backwash TSS	lbs/day	78	137	423	
Backwash BOD	mg/l	102	98	283	
Backwash BOD	lbs/day	47	82	254	
Backwash TKN	mg/l	0.4	2.0	3.0	
Backwash TKN	lbs/day	0.2	1.7	2.7	
Effluent Flow	mgd	1.04	1.91	3.47	4.40
Effluent TSS	mg/l	1.53	4.06	6.00	
Effluent TSS	lb/day	13.28	64.62	173.64	
Effluent TSS	kg/day	6	29	79	
Effluent BOD	mg/l	2.28	4.22	7.00	
Effluent BOD	lb/day	20	67	203	
Effluent BOD	kg/day	9	31	92	
Effluent NH3	mg/l	0.41	2.00	3.00	
Effluent NH3	lb/day	4	32	87	
Effluent NH3	kg/day	2	14	39	

LONG HILL TOWNSHIP WWTP
FLOW AND MASS BALANCE EVALUATION
EXISTING FLOW CONDITIONS

	UNITS	ANNUAL AVERAGE DAY	MAXIMUM MONTH	MAXIMUM DAY	PEAK HOURLY FLOW
POST AERATION SYSTEM					
Number of Tanks Installed		2	2	2	2
Volume per Tank	cf	3,151	3,151	3,151	3,151
Total Volume	cf	6,302	6,302	6,302	6,302
Total Volume	gal	47,145	47,145	47,145	47,145
Hydaulic Detention Time	hrs	1.1	0.6	0.3	0.3
D.O. concentration of filtered effluent	mg/L	1	1	1.5	1.5
Desired D.O.concentraiton of final effluent	mg/L	7.0	7.0	6.5	6.5
Acutal Oxygen Requirement	lb/day	52	96	145	184
Water temperature	degrees C	25	25	25	25
Saturation DO	mg/L	8.02	8.02	8.02	8.02
alpha		0.80	0.80	0.80	0.80
beta		0.95	0.95	0.95	0.95
Standard Oxygen Requirement	lb/day	324	595	557	707
Diffuser type		coarse	coarse	coarse	coarse
Oxygen transfer efficiency	%	12.00	12.00	12.00	12.00
Oxygen required	lb/day	2,702	4,962	4,644	5,889
Oxygen content of air	%	23.00	23.00	23.00	23.00
Weight of oxygen	lb/cf	0.017	0.017	0.017	0.017
Total air required	cf/day	158,939	291,897	273,185	346,401
Required blower capacity	cfm	110	203	190	241
Blower capacity	cfm	220	220	220	220
UV DISINFECTION					
# of UV Modules per Channel		5	5	5	5
Lamps per Module		4	4	4	4
# of Channels		2	2	2	2
Channel Width	ft	23.75	23.75	23.75	23.75
Manufacturer's stated capactiy	mgd	3.6	3.6	3.6	3.6
NJDEP Required Safety Factor	%	125	125	125	125
Actual Safety Factor based on flow				101%	79%
SOLIDS PRODUCTION SUMMARY					
WAS Production	lbs/day	589	928	2,368	
WAS Solids Content	%	0.74%	1.47%	3.44%	
WAS Flow	gpd	9,527	7,571	8,259	
TOTAL SLUDGE FLOW TO STORAGE					
Total Sludge Production	lbs/day	589	928	2,368	
Sludge Solids Content	%	0.74%	1.47%	3.44%	
Daily Average Sludge Flow	gpd	9,527	7,571	8,259	
Maximum Wastte Sludge Pumping Rate	gpm	69	55	60	

LONG HILL TOWNSHIP WWTP
FLOW AND MASS BALANCE EVALUATION
FUTURE FLOW CONDITIONS

	UNITS	ANNUAL AVERAGE DAY	MAXIMUM MONTH	MAXIMUM DAY	PEAK HOURLY FLOW
INFLUENT FLOWS & LOADS					
Influent Flow	mgd	1.24	2.27	4.14	5.24
Influent TSS Concentration	mg/l	117	113	198	
Influent TSS Load	lbs/day	1,210	2,149	6,841	
Influent BOD Concentration	mg/l	99	103	121	
Influent BOD Load	lbs/day	1,024	1,645	4,190	
Influent TKN Concentration	mg/l	18	16	16	
Influent TKN Load	lbs/day	186	299	514	
INFLUENT + RECYCLE FLOWS & LOADS					
Influent Flow	mgd	1.31	2.39	4.27	5.40
Influent TSS Concentration	mg/l	120	116	206	
Influent TSS Load	lbs/day	1303	2311	7346	
Influent BOD Concentration	mg/l	99	87	126	
Influent BOD Load	lbs/day	1080	1742	4493	
Influent TKN Concentration	mg/l	17	15	15	
Influent TKN Load	lbs/day	186	301	517	
OXIDATION CHANNELS					
Influent Flow	mgd	1.31	2.39	4.27	5.40
Influent TSS	lbs/day	1303	2311	7346	
Influent TSS	mg/l	120	116	206	
Influent BOD	lbs/day	1080	1742	4493	
Influent BOD	mg/l	99	87	126	
Influent TKN	lbs/day	186	301	517	
Influent TKN	mg/l	17	15	15	
Total Volume Installed	ft^3	121,249	121,249	121,249	121,249
Total Volume Installed	gals	907,000	907,000	907,000	907,001
Total Volume in Service	ft^3	121,249	121,249	121,249	121,249
Total Volume in Service	gals	907,000	907,000	907,000	907,001
Typical Design Criteria					
Hydraulic D.T. @ Design Flow (NJDEP)	hrs	7.5	7.5	N/A	N/A
Hydraulic D.T. @ Design Flow (WEF)	hrs	8-36	8-36	N/A	N/A
BOD Loading @ Design Flow(NJDEP)	lb/Kcf/day	38	38	N/A	N/A
BOD Loading @ Design Flow(10 STATES)	lb/Kcf/day	15	15	N/A	N/A
BOD Loading @ Design Flow(WEF)	lb/Kcf/day	5-30	5-30	N/A	N/A
Solids Retention Time @ Design Flow (WEF)	days	10-30	10-30	N/A	N/A
F/MLVSS Ratio at Desgin Flow (M&E Wastewater Engineerin Text)		0.04-0.1	0.04- 0.1	N/A	N/A
Actual Hydraulic Detention Time	hrs	16.7	9.1	5.1	4.03
Actual BOD Loading	lb/Kcf/day	9	14	37	
Solids Retention Time	days	20	16	9	
MLSS	mg/l	2,059	2,684	3,907	
MLSS	lbs	15,577	20,299	29,554	
Percent MLVSS	%	75%	75%	75%	
MLVSS	lbs	11,682	15,224	22,166	
F/MLSS		0.07	0.09	0.15	
F/MLVSS		0.09	0.11	0.20	
Sludge Yield (lb WAS/lb BOD removed)	lb/lb	0.70	0.70	0.70	
Biological Waste Sludge Production	lbs/day	703	1,105	2,821	
OXYGEN REQUIREMENTS					
Oxygen Required/BOD	lb/lb	1.3	1.3	1.3	
Influent TKN	mg/l	17	15	15	
Influent TKN	lbs/day	186	301	517	
Effluent NH3	mg/l	0.41	2.00	3.00	
Effluent NH3	lbs/day	4	40	107	
Oxygen Required/TKN	lbs/day	4.57	4.57	4.57	
Carbonaceous Oxygen Demand	lbs/day	1,305	2,053	5,239	
Nitrogenous Oxygen Demand	lbs/day	831	1,193	1,877	
Actual Oxygen Requirement (AOR)	lbs/day	2,135	3,246	7,116	
Operating DO	mg/l	2.0	2.0	0.9	
Water Temperature	°C	25	25	25	
Saturation DO	mg/l	8.02	8.02	8.02	
alpha		0.80	0.80	0.80	
beta		0.98	0.98	0.98	
Standard Oxygen Requirement (SOR)	lbs/day	3,674	5,585	10,307	
Standard Oxygen Requirement (SOR)	lbs/hr	153	233	429	
Oxidation Ditch #1 Brush Aerator Capacity @ 6.6 lb//hr/ft of shaft	lbs/hr	185	185	185	
Oxidation Ditch #2 Brush Aerator Capacity @ 5.85 lb/hr/ft of shaft	lbs/hr	245	245	245	
Total Oxidaiton Ditch Brush Aerator Capactiy	lbs/hr	430.00	430.00	430.00	

LONG HILL TOWNSHIP WWTP
FLOW AND MASS BALANCE EVALUATION
FUTURE FLOW CONDITIONS

	UNITS	ANNUAL AVERAGE DAY	MAXIMUM MONTH	MAXIMUM DAY	PEAK HOURLY FLOW
FINAL CLARIFIERS					
Number of Tanks Installed		2	2	2	2
Number of Tank in Service		2	2	2	2
Clarifier Diameter	ft	50	50	50	50
Clarifer Depth	ft	11.66	11.66	11.66	11.66
Area per Clarifier	sf	1,963	1,963	1,963	1,963
Total Area in Service	sf	3,927	3,927	3,927	3,927
Volume per Clarifier	cf	22,894	22,894	22,894	22,894
Total Volume in Service	cf	45,789	45,789	45,789	45,789
Volume per Clarifier	gals	171,273	171,273	171,273	171,273
Total Volume in Service	gals	342,545	342,545	342,545	342,545
Design Criteria:					
Overflow Rate @ Design Flow (NJDEP)	gpd/sf	<1000	<1000	N/A	N/A
Overflow Rate @ Peak Flow (10 STATES)	gpd/sf	N/A	N/A	<1,000	<1,000
Overflow Rate @ Avg & Peak (WEF)	gpd/sf	400-700	400-700	1000-1600	1000-1600
Solids Loading Rate (10 STATES)	lbs/sf day	N/A	N/A	<35	N/A
Solids Loading Rate (WEF) *Solids Flux Analysis	lbs/sf day	SF*	SF*	SF	
Solids Loading Rate (M&E Wastewater Engineering Text)	lbs/sf day	12-24	12-24	<34	N/A
RAS Flow % of Influent Flow (M&E Wastewater Engineering Text)	%	75 to 150	75 to 150	N/A	N/A
Solids Flux Capacity at RAS flow and SVI between 100 and 150	lbs/sf day	15 to 20	24 to 30	39 to 51	
Actual Overflow Rate	gpd/sf	333	608	1,087	1,376
Actual Solids Loading	lbs/sf day	8	20	44	
RAS Pumping System Capacity	mgd	1.2	1.2	1.2	
RAS Flow	mgd	0.6	1.2	1.2	
Underflow Rate (RAS flow divided by clarifer surface area)	gpd/sf	152.79	305.58	305.58	
RAS % of Influent Flow	%	46%	50%	28%	
RASS	mg/l	6,286	7,796	17,069	
RASS	lbs/day	31,076	77,088	168,780	
MLSS	mg/l	2,059	2,684	3,907	
Total Flow (Plant + RAS)	mgd	1.84	3.47	5.34	
MLSS Load	lbs/day	31,634	77,661	174,002	
WAS Production	lbs/day	703	1,105	2,821	
WAS Solids Content	mg/l	6,286	7,796	17,069	
WAS Solids Content	%	0.63%	0.78%	1.71%	
WAS Flow	mgd	0.013	0.017	0.020	
WAS Flow	gpd	13,401	16,999	19,817	
Clarifier Effluent Flow	mgd	1.31	2.39	4.27	5.40
Clarifier Effluent TSS	mg/l	10.00	12.00	20.00	20
Clarifier Effluent TSS	lb/day	109.03	239.14	711.91	
Clarifier Effluent TSS	kg/day	49	108	323	
Clarifier Effluent BOD	mg/l	7.00	8.20	13.00	
Clarifier Effluent BOD	lb/day	76	163	463	
Clarifier Effluent BOD	kg/day	35	74	210	
Clarifier Effluent NH3	mg/l	0.41	2.00	3.00	
Clarifier Effluent NH3	lb/day	4	40	107	
Clarifier Effluent NH3	kg/day	2	18	48	
FILTERS					
Number of Continuous Backwash Filters		4	4	4	4
Total surface area per filter	SF	150	150	150	150
Total Filter Surface Area	SF	600	600	600	600
Design Criteria					
Filtration Rate (10 States)	gpm/sf	N/A	N/A	N/A	<5
Filtration Rate (M&E Wastewater Engineering Test)	gpm/sf	2	2	<5	<5
Filtration Rate (Manufacturer)	gpm/sf	2 to 3	2 to 3	<5	<5
Maximum TSS concentration (Manufacturer)	mg/L	20 to 30	20 to 30	20 to 30	20 to 30
Actual Filtration Rate	gpm/sf	1.51	2.77	4.94	6.25
Acutal Flter Peak Floow Capacity 2.8 mgd = 3,2 gpm/sf					
Recycle Flow:					
Backwash flow % of Forward Flow	%	5	5	3	3
Backwash flow	mgd	0.07	0.12	0.13	0.16
Backwash TSS	mg/l	171	163	472	
Backwash TSS	lbs/day	93	162	505	
Backwash BOD	mg/l	102	98	283	
Backwash BOD	lbs/day	56	97	303	
Backwash TKN	mg/l	0.4	2.0	3.0	
Backwash TKN	lbs/day	0.2	2.0	3.2	
Effluent Flow	mgd	1.24	2.27	4.14	5.24
Effluent TSS	mg/l	1.53	4.06	6.00	
Effluent TSS	lb/day	15.86	76.80	207.17	
Effluent TSS	kg/day	7	35	94	
Effluent BOD	mg/l	2.28	4.22	7.00	
Effluent BOD	lb/day	24	80	242	
Effluent BOD	kg/day	11	36	110	
Effluent NH3	mg/l	0.41	2.00	3.00	
Effluent NH3	lb/day	4	38	104	
Effluent NH3	kg/day	2	17	47	

LONG HILL TOWNSHIP WWTP
FLOW AND MASS BALANCE EVALUATION
FUTURE FLOW CONDITIONS

	UNITS	ANNUAL AVERAGE DAY	MAXIMUM MONTH	MAXIMUM DAY	PEAK HOURLY FLOW
POST AERATION SYSTEM					
Number of Tanks Installed		2	2	2	2
Volume per Tank	cf	3,151	3,151	3,151	3,151
Total Volume	cf	6,302	6,302	6,302	6,302
Total Volume	gal	47,145	47,145	47,145	47,145
Hydaulic Detention Time	hrs	0.9	0.5	0.3	0.2
D.O. concentration of filtered effluent	mg/L	2	2	2.0	2
Desired D.O.concentraiton of final effluent	mg/L	7.0	7.0	6.5	6.5
Acutal Oxygen Requirement	lb/day	52	95	155	197
Water temperature	degrees C	25	25	25	25
Saturation DO	mg/L	8.02	8.02	8.02	8.02
alpha		0.80	0.80	0.80	0.80
beta		0.95	0.95	0.95	0.95
Standard Oxygen Requirement	lb/day	199	365	502	636
Diffuser type		coarse	coarse	coarse	coarse
Oxygen transfer efficiency	%	12.00	12.00	12.00	12.00
Oxygen required	lb/day	1,662	3,038	4,187	5,300
Oxygen content of air	%	23.00	23.00	23.00	23.00
Weight of oxygen	lb/cf	0.017	0.017	0.017	0.017
Total air required	cf/day	97,780	178,712	246,314	311,760
Required blower capacity	cfm	68	124	171	217
Blower capacity	cfm	220	220	220	220
UV DISINFECTION					
# of UV Modules per channel		5	5	5	5
Lamps per Module		4	4	4	4
# of Channels		2	2	2	2
Channel Width	ft	23.75	23.75	23.75	23.75
Manufacturer's stated capactiy	mgd	4.4	4.4	4.4	
NJDEP Required Safety Factor	%	125	125	125	125
Actual Safety Factor based on flow		337%	184%	103%	
SOLIDS PRODUCTION SUMMARY					
WAS Production	lbs/day	703	1,105	2,821	
WAS Solids Content	%	0.63%	0.78%	1.71%	
WAS Flow	gpd	13,401	16,999	19,817	
TOTAL UUNTHICKED SLUDGE FLOW TO STORAGE					
Total Sludge Production	lbs/day	703	1,105	2,821	
Sludge Solids Content	%	0.63%	0.78%	1.71%	
Daily Average Sludge Flow	gpd	13,401	16,999	19,817	
Maximum Wastte Sludge Pumping Rate	gpm	98	124	144	